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SEISMIC STABILITY EVALUATION OF ALBEN BARKLEY LOCK AND DAM PROJECT

Volume 4

LIQUEFACTION SUSCEPTIBILITY EVALUATION AND POST-EARTHQUAKE STRENGTH DETERMINATION

by

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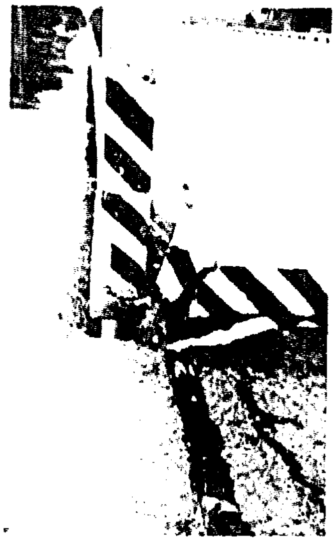

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This report documents the results of seismological, geological, laboratory, field, and analytical investigations conducted to evaluate the liquefaction potential of two earth embankment sections of the Alben Barkley Lock and Dam Project, Kentucky. These sections are representative of those of the main embankment and powerhouse/switchyard areas. The design earthquake, from the New Madrid Seismic Zone, had a body-wave magnitude of 7.5. Of particular interest in this study was the evaluation of the liquefaction potential of silty sands in the foundation.

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PREFACE

The US Army Engineer Waterways Experiment Station (WES) was authorized to conduct this study by the US Army Engineer District, Nashville (ORN), by Intra-Army 'rder for Reimbursable Services Nos. 77-31 and 77-112. This report is Volume 4 of a 5-volume set which documents the seismic stability evaluation of Alben Barkley Dam and Lake Project. The 5 volumes are as follows:

Volume 1: Summary Report

Volume 2: Geological and Seismological Evaluation

Volume 3: Field and Laboratory Investigations

Volume 4: Liquefaction Susceptibility Evaluation and Post-Earthquake Strength Determination

Volume 5: Stability Evaluation of Geotechnical Structures

The work discussed in this volume is a joint endeavor between ORN and WES. Mr. Paul F. Bluhm, of the Geotechnical Branch (ORNED-G) at ORN, coordinated the contributions from ORN. Mr. Ronald E. Wahl of Soil and Rock Mechanics Division, Richard S. Olsen and Dr. M. E. Hynes of the Earthquake Engineering and Geophysics Division (WESGG-H) at WES, coordinated the work by WES. The preliminary stages of this project were conducted by Dr. William F. Marcuson III, who was Principal Investigator from 1976 to 1979. From 1979 to 1988, Dr. M. E. Hynes was Principal Investigator. Mr. Wahl was Principal Investigator from 1988 to project completion. Significant engineering support was provided by Mr. Donald E. Yule of WESGG-H. Additionally, Mr. Daniel Habeeb, Mr. Melvin Seid, and Ms. Charlotte Caples provided valuable assistance in the preparation of this report.

Overall direction at WES was provided by Dr. A. G. Franklin, Chief, WESGH, and Dr. Marcuson, Chief, Geotechnical Laboratory.

Overall direction at ORN was provided by Mr. James E. Paris, Chief, Soils and Embankment Design Section, Mr. Marvin D. Simmons, Chief, Geology Section, and Mr. Frank B. Couch, Jr., Chief, Geotechnical Branch. Mr. E. C. Moore was Chief, Engineering Division. Former District Commanders during the study were COL Robert K. Tenner, COL Lee W. Tucker, and COL William K. Kirkpatrick, COL Edward A. Starbird, and COL James P. King. The current District Commander 's LTC Stephen M. Sheppard.

Technical Advisors to the project were the late Professor H. B. Seed (University of California, Berkeley), Professors Alberto Nieto (University of

Illinois, Champaigne- Urbana) and L. Timothy Long (Georgia Institute of Technology), and Dr. Gonzalo Castro (Geotechnical Engineers, Inc.).

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acre-feet	1,233.489	cubic metres
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
feet per mile	0.1893935	metres per kilometer
inches	2.54	centimetres
kips (force) per square foot	47.88026	kilopascals
miles (US statute)	1.609347	kilometres
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
square miles	2.589998	square kilometres
yards	0.9144	metres

SEISMIC STABILITY EVALUATION OF ALBEN BARKLEY DAM AND LAKE PROJECT

LIQUEFACTION SUSCEPTIBILITY AND POST-EARTHQUAKE STRENGTH DETERMINATION

PART I: INTRODUCTION

Background

1. This report is one of a series of five reports which document the investigations and results of a seismic stability evaluation of the Alben Barkley Dam and Lake Project, located on the Cumberland River, approximately 25 miles upstream of Paducah, Kentucky. This seismic safety evaluation was performed as a cooperative effort between the US Army Engineer Waterways Experiment Station (WES) and the US Army Engineer District, Nashville (ORN), and in accordance with Engineering Regulation 1110-2-1806, dated 16 May 1983. Construction of the Barkley Project began in 1957 and was completed in 1966. As a key unit in the comprehensive plan of the development of the Cumberland River, the multipurpose Barkley Project provides flood control, hydroelectric power, navigation, and recreational facilities. The reservoir is contained by a concrete gravity section flanked by earth embankment dams. The concrete gravity section includes a gated spillway, a lock, and a powerhouse. The dam supports a railroad track system which traverses most of the dam crest. A canal, large enough for barge traffic, connects Barkley and Kentucky Lakes about 2.5 miles upstream from the dam. A location map and plan view of the project are shown in Figure 1. The reader is referred to Volume 3 of this series for a more detailed description of the site.

2. At the maximum flood control pool, Elevation 375 ft, the reservoir stores 2,082,000 acre-feet of water, with 13 ft of freeboard (minimum crest Elevation 388 ft). For normal operation, the pool elevation varies from 354 to 359 ft, and stored volume varies from 610,000 to 869,000 acre-feet, respectively. The maximum pool of record over the period 1966 to 1987 was 370.04 ft (on 13 May 1989). The guide curve for the reservoir levels is shown in Figure 2. A pool elevation of 360 ft was used for the seismic stability

evaluation. A pool elevation of 360 ft leaves an available freeboard of 28 ft at the time the design earthquake is assumed to occur.

3. The geological and seismological investigations for the project are documented in Volume 2 of this series (Krinitzsky 1986). The most severe seismic threat was determined to be an earthquake of body-wave magnitude (m_b) of 7.5, at a distance of about 118 km, in the New Madrid source zone. The S48°E component of the Santa Barbara Courthouse record from the Kern County, CA earthquake of 21 July 1952 was scaled to a horizontal peak acceleration of 0.24 g to represent this design earthquake. The resulting peak velocity was 35 cm/sec and the duration above 0.05 g was about 60 sec.

4. The concrete gravity dam, powerhouse, and lock system are 109 ft tall at the maximum section and are founded on a limestone bedrock. The embankment dams are founded on an alluvial deposit with a maximum thickness of approximately 120 ft. The alluvium is underlain by the limestone bedrock. This alluvium, a complex system of interbedded clays, silty sands, and gravels, is the focus of concern in the seismic safety assessment due to the possibility of liquefaction during the design earthquake. Generally, the alluvium beneath the embankment can be viewed as consisting of three units as shown on the profile in Figure 3. The first zone, Unit 1, extends from the ground surface to a depth of 10 to 20 ft and is generally made up of a medium stiff clay with low to moderate plasticity. This material is an overbank deposit laid down on the flood plain during flooding. The second zone, Unit 2, extends from the bottom of Unit 1 to a depth of about 50 ft and consists of a highly stratified sequence of clays, sands, and, silty sands. These are overbank deposits whose interbedded layers vary widely with respect to grain size and thickness. Unit 3 extends from the bottom of Unit 2 to a depth of 120 ft and consists of gravels and denser sands (denser than Unit 2) with some clay also being present. These materials are channel deposits laid down as the river meandered across the valley. The different depositional environments for each of the three units described above probably resulted from changing base levels that occurred in the geologic past. These deposits range widely in grain size.

Principal Objectives and Scope of Work

5. The principal objective of this project was to evaluate the performance of the alluvial foundation underlying two specific areas of the right embankment dam in the event of the design earthquake. The first area studied was that of the switchyard located between Sta 38+00 (at the interface of the concrete dam and earth embankment) and Sta 43+00. The second area of interest was that of the main embankment, located between Sta 43+00 and Sta 88+00. The stationing used in this report are shown on the cross sectional and plan views of Figures 4 and 5, respectively.

6. The specific tasks undertaken and reported herein include:

a. Characterization and stratigraphic evaluation of the foundation alluvium beneath the dam.

b. A site specific dynamic analysis to determine the response of the embankment and foundation to the design earthquake motions.

c. Evaluation of the potential for and extent of liquefaction in the alluvial soils. The evaluation was made based on a comparison of the cyclic strength and dynamic shear stresses induced by the earthquake.

d. Estimation of the post-earthquake strength of the embankment and foundation alluvium for a post-earthquake stability analysis.

7. The results of the field and laboratory investigations reported in Volume 3 provided input to the stratigraphy evaluation, the dynamic response analysis, and the determination of the post-earthquake strengths. The results of the post-earthquake stability analysis are reported in Volume 5 of this series.

PART II: STRATIGRAPHY EVALUATION AND SITE CHARACTERIZATION

General

8. A stratigraphy evaluation performed for characterizing and idealizing the site was an essential step in the seismic stability analysis of this dam. The stratigraphy evaluation described in this Part was based on data obtained from an extensive exploration program of the site which included field and laboratory tests. This program was reported in detail in Volume 3 of this series. The major sources of data upon which the stratigraphy evaluation is based included Standard Penetration Tests (SPT), Cone Penetration Tests (CPT), undisturbed soil sampling, and analysis of a downstream exposure of one of the dam's major foundation units.

9. It must be stressed that the stratigraphy of the foundation soils is complex and could only be evaluated by analysis of all the major sources of data rather than that from any one source alone. Consideration was given to the strengths and weaknesses of each source of information in order to develop as complete an understanding of the foundation conditions as possible. After a brief discussion of the foundation conditions, the following sections of this part of the report provide discussions of the factors from each data source which were weighed and used in evaluating the characteristics of the foundation soils for this study. These discussions are not presented in the sequence in which the various source of information were obtained, but are rather presented in an order intended to produce, as clearly as possible, a picture of the site's foundation conditions.

Description of Foundation Conditions

10. This section was excerpted from Volume 3 and briefly describes the major foundation units present in the subsurface. These descriptions provide a useful background with which to begin the discussion. Subsequent analysis of data documented later in this section will add some detail and refinement to the initial interpretation.

11. The foundation beneath Barkley Dam is complex and can be viewed as consisting of three basic units: Units 1, 2, and 3. Longitudinal cross-sections along the toe and centerline showing the three foundation units are

shown in Figures 3 and 4. Unit 1 is a medium stiff clay whose thickness varies between 20 and 30 ft. In the switchyard area (between Sta 38+00 and Sta 43+00) the thickness of Unit 1 extends to Elevation 320. In the area along the main embankment (between Sta 43+00 and 88+00) this unit is somewhat thinner where it only extends to Elevations between 330 and 325 ft. The exploration program revealed that some sand layers are present but the number and extent of these layers are very limited. The average liquid limit, plastic limit and natural water content in this unit are about 30, 17, and 23 percent respectively.

12. Unit 2, directly below Unit 1, extends to elevations approximately between 305 and 300 in the switchyard area. Beneath the main embankment Unit 2 lies a little deeper where it extends between the elevations 300 and 295. Unit 2 is dominated by a very soft clay which contains layers of interbedded sands and silty sands. These layers are frequently very thin, generally having thicknesses of less than 6 in. The SPT and CPT data collected during the field investigations showed that the sand (and clays) had low penetration resistances. The average fines content was determined to be about 28 percent based on laboratory tests of samples. The low penetration resistance indicated that the sandy layers had the potential for liquefaction and that the prediction of their seismic performance should be a major element in the evaluation of the dam's post-earthquake stability. This concern made it imperative to determine the lateral extent and continuity of these sand layers since the dimensions of materials with high liquefaction potential are important factors in evaluating the seismic stability of an embankment.

13. Foundation Unit 3 is made up of sands and gravels although some layers of clay are occasionally present. One continuous layer of clay was found in the switchyard and free field area between elevations 295 and 290 ft. The CPT and SPT data indicate that the sands and gravels of Unit 3 are denser than the sands in Unit 2. Also, the sands of Unit 3 are cleaner than those of Unit 2 and have an average fines content of about 15 percent. The liquefaction potential of the cohesionless materials in Unit 3 were also evaluated as part of this analysis.

Undisturbed Samples

14. Undisturbed samples were obtained from eleven borings which are listed in Table 1. The locations of these borings are shown in Figure 5. Undisturbed samples were obtained from each of the three foundation units for the purposes of estimating density, providing materials for laboratory strength tests (both cyclic and static), and to provide an opportunity to view the characteristics of the stratigraphy in the foundation. This section of the report is reserved for discussion concerning only the latter purpose. Discussions of the details of the other aspects of undisturbed sampling program are included in Volume 3. In the following paragraphs photographs of representative undisturbed samples from Units 2 and 3 are discussed and qualitatively illustrate some of the characteristic traits of the soils in these units.

15. Figures 6 and 7 contain photographs of two of the undisturbed samples taken from Boring DS-1. Boring DS-1 is located just off the downstream toe of the main embankment near Sta 65+00 as shown on Figure 5. Sample 14 (shown in Figure 6) shows the interbedding of sand and clay layers. The dark and light colored materials in the photograph represent the clay and sand layers in Unit 2, respectively. The photograph shows that the sand layers are very thin and do not exceed seven inches in thickness and appear to be interbedded within the clay material. This and other photographs show that this type of interbedding is a characteristic trait of Unit 2. More photographs of the undisturbed samples can be seen in Appendix G to Volume 3. It was not possible to tell whether or not the sand layers are continuous on the basis of the photographs of undisturbed samples.

16. Sample 20 from DS-1 shown in Figure 7 is considered to be a representative sample of the sands and gravels retrieved from Unit 3. This photograph (and others) shows that the sand is by far the dominate material in the sample and suggests that the sand layers in Unit 3 are a great deal thicker than those of Unit 2. Some thin clay layers (similar in appearance to those of Unit 2) are evident in other pictures from Unit 3 samples.

Stream Bank Excavation at Downstream Location

17. An excavation was performed along the streambank at a location downstream of the Barkley damsite in an exposure of Unit 2. The excavation site was located approximately two miles downstream from the dam. A more detailed presentation of the procedures, equipment, and results of the analysis of the excavation is provided in Volume 3.

18. In the analysis of data obtained from soil borings, it was not possible to map individual sand layers encountered in Unit 2 from one boring to the next thereby making it impossible to determine the continuity and lateral extent of these layers. The excavation was performed in hopes of overcoming this difficulty. The exposure and excavation are in the direction of river flow. From geological reasoning, it was presumed that individual soil layers would be more continuous in the direction of river flow which is perpendicular to the axis of the dam.

19. The exposed soils on the face of the excavation were mapped and photographed. The dimensions of the exposure in the excavation were about 30 ft in length by about 6 ft in height. The detailed geologic cross section of the exposed soils is shown on Figure 8. Two typical photographs of the exposed face of the excavation are presented in Figure 9. Both the map and the photographs yielded important information as to the complex stratigraphy present in the Unit 2 soils. Paraphrasing the findings of Volume 3, the analysis of these two items shows clearly that the clay is the dominant material in the unit. The sand beds have an average thickness of about 2 to 4 in. The maximum thickness observed in one of the sand beds was 1.5 ft. The lengths of the sand beds varied greatly, from just a few inches to greater than the 30 ft which is the length of the excavation. One bed, located outside the limits of the exposure, was traced for a distance of 150 ft. The evidence also shows that the sand beds undulate and horizontal planes of constant elevation do not remain in contact with any bed for more than a distance of about 20 ft. Based on these findings, it was concluded in Volume 3 that significant continuity may exist in the sand layers in the direction parallel to the river. Due to the orientation of the exposure it was not possible to determine the degree of continuity in the direction perpendicular to the river flow.

Standard Penetration Test Data

20. A comprehensive SPT program was performed at the damsite for the purpose of determining the penetration resistances in terms of blowcounts for the foundation units and also to obtain disturbed samples for soil classification and natural water content. The penetration resistances were later used as an index in the determination of liquefaction resistance as well as to aid in the evaluation of stratigraphy. The samples retrieved from SPT testing were also useful in evaluating the characteristics of the soils in the three foundation units and in evaluating the liquefaction potential of the foundation soils.

21. Forty-four SPT soundings were performed at the Barkley Dam during the period between 1977-1984. The SPT boring locations are shown in the plan of Figure 5. These tests employed various types of equipment and procedures which are discussed in Volume 3. The basic data gathered from these SPT borings included blowcounts, jar samples, and laboratory index tests (Atterberg Limits and grain size analysis) from each split spoon sample. This information was stored in a computerized database for referral during analysis.

22. Information stored in the database is included in Appendices A and B to Volume 3. Additionally, plots of depth versus measured SPT blowcount, grain size data, mean grain size, and fines contents were developed to aid evaluation of the liquefaction potential and to assist in the stratigraphic evaluation.

23. Data obtained from boring BEQ-10 are considered typical of the SPT data gathered from the three foundation units at the site. Boring BEQ-10 is located just off the downstream toe of the main embankment near Station 54+00. The SPT plot for boring BEQ-10 is shown in Figure 10. Similar plots for all SPT borings are presented in Appendix A. Table 2 is a companion to the plot and lists data obtained from laboratory index tests performed on split spoon (disturbed) samples of this boring. Superimposed on the left hand side of Figure 10 are the locations of boundaries of the three foundation Units 1, 2, and 3.

24. The measured penetration resistances (blowcounts) from borings located along the downstream toe of the main embankment are plotted on the cross section shown in Figure 11. The data on the cross section and the soil classifications from the database were used to develop the boundaries between

the three foundation units and to help characterize the soils within each unit. Examination of the data shows that each unit has certain characteristics which distinguish it from the others. These characteristics will be discussed in the following paragraphs.

25. Unit 1 is characterized by measured blowcounts which have a general tendency to decrease with depth. The cross section shows that Unit 1 extends from the ground surface to approximately Elevation 325 ft. The classifications of soil samples in the reach show (See Appendix A to Volume 3) that the Unit 1 materials are clays which classify as CL materials though some sand is present. The tendency for blowcounts to decrease with depth suggests that these clays are overconsolidated due to desiccation.

26. Unit 2, situated mainly between Elevations 325 and 295 in the area of the main embankment, contains the interbedded sands and clays which were discussed earlier. Figure 11 shows that the measured blowcounts in Unit 2 are relatively low, typically between 5 and 15. Matching the soil classification data up with the blowcount data at corresponding depths shows that the blowcounts with the higher values will nearly always be associated with sands while the lower values tend to be associated with the clayey materials. The sands in the clayey materials generally classified as SM materials and the clays generally classified as CL. A histogram showing the distribution of mean grain size (D_{50} in mm) for samples recovered in Unit 2 between Elevations 320 and 305 is shown on Figure 12. This elevation range was selected to represent Unit 2 throughout this study even though some variations occur at various locations about the site. This plot shows that the average D_{50} is about 0.15 mm. A histogram of the percentage passing the No. 200 sieve for sands, shown on Figure 13, shows a fairly uniform distribution over the range 0 to 50 percent. The mean value is about 30 percent which indicates that the sands in Unit 2 are fairly dirty. Examination of the database also shows that the fines are non-plastic. The distributions include samples retrieved from all SPT borings at the site including those in the switchyard area.

27. Unit 3 is located below about elevation 295 in the area of the main embankment. Unit 3 consists mainly of sands and gravels which are characterized by higher penetration resistances than those observed in the sands in Unit 2. This indicates that the Unit 3 sands are denser than those in Unit 2. The measured penetration resistance is generally above 30 blows/ft which is significantly greater than that of the sands in Unit 2. The histogram in

Figure 14 of mean grain size for the sandy materials in Unit 3 shows that the D_{50} is about 0.20 mm. These are only slightly coarser than the D_{50} of the Unit 2 samples. A histogram showing the percentage of fines is shown on Figure 15. This plot shows the average fines content of these sands is about 10 percent and indicates that the sandy materials of Unit 3 are cleaner than those of Unit 2. The database shows that the fines of Unit 3 sands are also non-plastic. The database showed that 344 sand samples were recovered from Unit 3 while only 162 sand samples were recovered from Unit 2. These totals translate to 250 jar samples recovered per 1000 ft of drilling in Unit 2 compared to 970 jar samples recovered per 1000 ft in Unit 3. These figures suggest that there is a significantly greater amount of sand in Unit 3 than in Unit 2. Most of the SPT borings in Unit 3 terminated at depths well before bedrock was encountered.

28. The discussions of the previous paragraphs indicate that the SPT borings provided some very useful information regarding the characteristics of the three foundation units at the damsite. The principle advantage in using the SPT on this project was the fact that it provided hundreds of samples from which the component soils of each foundation unit could be classified. Soil classification is important in evaluating the liquefaction potential of a deposit as different soil types respond differently to earthquake shaking. The soil classifications also assisted greatly in assessing the stratigraphy. However, the SPT has several limitations as well, particularly with respect to its use in Unit 2 where the thin sand layers interbedded within the clay are suspected of having a high liquefaction potential. The SPT blowcount data does not provide a sufficiently high enough degree of resolution to clearly assess all the sand layers encountered by the boring. This is because the blowcounts are taken over a one foot depth interval. A one-foot drive distance is greater than the thickness of most of these layers which are only a few inches thick. Thus, the measured penetration resistance includes a contribution from the softer clay. Also, many of the sand layers went undetected due to the thinness of the layers and the fact that the penetration resistance observed while the split-spoon is actually in the sand is influenced by the softer underlying clays. It was also impossible to correlate the continuity of individual sand layers from one boring to the next. These limitations were less of a problem in Unit 3 where the sand layers were thicker.

Cone Penetration Test (CPT) Test Results

29. Sixty-five Cone Penetration Tests were performed in the switchyard area for the purpose of estimating strength and evaluating the stratigraphy of the foundation soils. The estimation of cyclic and post-earthquake strengths from the CPT data are discussed in Parts IV and V of this report. This section is reserved for the discussion of how the CPT data were used to evaluate the stratigraphy of the foundation soils. The location of the CPT soundings is shown in the plan of Figure 16. The locations of these tests were concentrated in the switchyard area because the preliminary liquefaction and slope stability studies indicated that the switchyard area was the most critical area in terms of the dam's seismic stability. As with the other types of insitu tests performed at the damsite the details concerning the equipment and conduct of the CPT tests at the damsite are included in Volume 3.

30. The advantages of the CPT include its ability to provide a continuous record of data and to resolve stratigraphic changes with a resolution of a few inches. These advantages were particularly useful at Barkley Dam in light of the complex foundation conditions there. Additionally, the CPT is relatively low in cost when compared with the more conventional SPT and undisturbed sampling programs. The principle disadvantage of the CPT is the fact that there is no sample recovery. However, this disadvantage was offset to a large degree by the fact that samples recovered from the SPT and undisturbed sampling program (discussed previously) proved to be useful supplements to the CPT data gathered from the Barkley site.

31. The CPT's advantages are illustrated in Figure 17. In this figure, the CPT penetration resistance (cone tip resistance measured in tsf) is compared with the SPT penetration resistance (blowcounts). Data from Boring BEQ-26 and CPT-36, located in close proximity to each other near Sta 39+50 (See Figure 16), were selected for the comparison. The measured SPT blowcounts for BEQ-26 are shown on the left side of the figure and the cone tip resistances (in TSF) are shown on the right. The CPT penetration resistance is represented by two traces: the measured (raw) cone resistance, q_c, and the corrected cone resistance (adjusted to overburden stresses of 1 tsf), q_{cn}. Clearly, the CPT reveals more detail than the SPT. This is especially true in Unit 2 where the sand layers are detected as spikes of relatively high penetration resistance which are superimposed on the low penetration resistance

background of Unit 2's soft clays. The width of the spikes is an indication of the thickness of the sand layer. Conversely, a detailed manifestation of the sand layers is not revealed by the SPT blowcounts; however, the sands were sampled by the split spoon and identified in the lab analysis. The lack of detail in the SPT data is an outcome of the discontinuous nature of the testing procedure. The thin sand layers often have a negligible effect over the 18 in. drive intervals of the blowcounts. Thus, if a sand layer is very thin its contribution to the blowcount may be insignificant. For both SPT and CPT, the true penetration resistances of the thin sands may be underestimated due to the effect of the underlying soft clay which strains before being encountered by the penetrometer. The effect is greater for the SPT split spoon sampler than for the cone penetrometer since the split-spoon sampler has an outside diameter of 2 in. which is greater than the 1.40 in. diam of the cone.

32. The CPT test can also be used as an aid to classify soils. A classification scheme devised by Olsen (1984) was used to identify the types of soils encountered by the CPT probe. The procedure which correlates basic CPT data to soil types is discussed in this paper. The chart used in the CPT soil classification scheme is shown in Figure 18. The basic idea behind Olsen's system is that basic soil types can be identified by the combinations of corrected values of sleeve friction and cone resistance, f_{sn} and q_{cn}. The corrected parameters, f_{sn} and q_{cn}, are the results of adjustments made to the measured values of sleeve friction and cone resistance, f_s and q_c. These adjustments correct the measured values to overburden stress conditions of 1 TSF. The adjusted CPT parameters are mapped onto the soil classification chart with the outcome being a Soil Characterization Number (SCN) which correlates to basic soil types. A CPT SCN of 0.5 is a typical clay, the range of SCN of 1 to 2 represents silt mixtures, SCN's for sands range from 2 to 4, and a fine sand has a SCN between 2.5 and 3.5. The SCN numbers for various combination of normalized sleeve and cone tip resistances are also shown on Figure 18.

33. Data from CPT-36, shown in Figure 19, were used as an example of the application of the CPT soil classification. The left and center panels show the cone and sleeve readings (both measured and corrected) which were used to enter the chart shown in Figure 18 to determine the SCN's and soil types which are shown in the right panel.

34. Charts similar to that in Figure 19 were made up for each of the 65 CPT soundings at the damsite. These are included in Appendix B to this report. The CPT cone resistance and soil classification charts were used to evaluate the stratigraphy in the switchyard area by developing cross sections along lines of CPT soundings. Three cross sections were developed from lines of CPT shown on the plan in Figure 15. These cross sections were developed along three lines: B'-B' which is parallel to the axis of the dam and runs along the toe of the switchyard berm, A'-A' which is also parallel to the axis and runs along the top of switchyard berm at Elevation 366 ft, and D'-D' which is perpendicular to the axis of the dam. Cross sections displaying the cone resistances, q_c and q_{cn}, and CPT soil classifications were prepared for each of these lines. The cone resistance and soil classification cross sections for line D'-D' (perpendicular to axis) are shown on Figures 20 and 21, respectively. Those for line B'-B' (along toe of switchyard berm) are shown on Figures 22 and 23, and those for A'-A' are shown on Figures 24 and 25.

35. From analysis of these sections a refined interpretation was made of the foundation stratigraphy of the damsite. In Unit 1 the cone resistances have a general tendency to decrease with depth. Since the CPT is an index of undrained shear strength, this feature indicates that the clays are likely to be overconsolidated due to desiccation. The cross sections also show that some sands are present. The cross sections show that the boundary between Unit 1 and Unit 2 undulates to some degree but generally the boundary lies between Elevations 325 and 320 in the switchyard area.

36. In Unit 2, CPT cone resistances profiles are characterized by spikes superimposed upon relatively low penetration resistance as was discussed earlier. The spikes are indicative of sands and the low values are indicative of clays. The points of the q_{cn} trace for the clays aline themselves in a nearly vertical line which indicates that the clays in Unit 2 are normally consolidated. The CPT classifications alternate between sand mixtures and clays and show that the clays are the dominant material in Unit 2. This is consistent with the laboratory classifications of SPT samples. It was not possible to correlate the continuity of the individual sand layers from sounding to sounding in Unit 2 from the CPT data. The boundary between Units 2 and 3 undulates to a minor degree in both directions but generally lies between elevations 305 and 300 in the switchyard area only. The CPT data reveal that Unit 3 is generally made up of sandy materials with some

interbedded clays. Unit 3 was subdivided into three basic zones of materials t ed on analysis of the CPT data: Units 3a, 3b, and 3c. The interpreted boundaries of each of are shown on Figures 20 through 25. In general there is a marked increase in penetration resistance as the probe crosses the boundary between Units 2 and 3a. The increase is due to an increase in density in the sands and the presence of gravel of Unit 3a. The CPT classification suggests that the Unit 3a sands are cleaner than those of Unit 2 as evidenced by the SCN which is typically much higher for the Unit 3 sands. This is consistent with the grain size data from the SPT sand samples. The CPT data shows that the Unit 3a sand layers are probably much thicker than the Unit 2 sand layers which is another possible reason for their higher penetration resistance values.

37. A zone of low cone resistance, designated as Unit 3b, was generally detected between the elevations of 295 ft and 288 ft. The cone resistance profiles of this zone have an appearance which is remarkably similar to those of Unit 2 with some thin sand lenses interbedded within the clay. This zone is extensive as it was detected by nearly all of the CPT soundings performed in the switchyard area, therefore it was treated as a characteristic of the site in the switchyard area. The foundation materials below the low blowcount zone were designated Unit 3c and have characteristics similar to those of Unit 3a.

38. A technique was developed during the project which uses the CPT data to quantify the percentage of sand present for any elevation across the site. The estimate was made by dividing the foundation into 0.1 ft intervals between elevations 283 and 350. The CPT database was queried to determine the SCN of the material occupying each elevation interval for each sounding. In this analysis, an interval was considered to be occupied by a sand if the SCN value was greater than two. Thus, an area-wide percentage of sand at a particular elevation was estimated by computing the ratio of the number of times the CPT probe encountered a sandy material (having a SCN > 2) divided by the total number of CPT soundings passing through that elevation interval.

39. Sand contents were estimated for the switchyard area, the free field area just downstream of the switchyard area, and the combined total for both areas. The results are shown in Figure 26. The charts show that the normally consolidated clays discussed earlier are by far the dominant material in Unit 2 (approximately between elevations 320 and 305 ft) and that the sands

make up less than 20 percent of the material in this unit. There is even less sand in Unit 2 in the switchyard area than in the free field area. Sand is the dominant material comprising about 60 percent of the material in Unit 3 (below 305). In the switchyard, the presence of Unit 3b is detected between elevations 288 and 293 where the sand percentage decreases. The sand percentage in the switchyard increases in Unit 3c below elevation 288.

Summary and Results of Stratigraphy Evaluation

40. Data collected from SPT, CPT, undisturbed samples, and an excavated expcsure of a foundation unit were analyzed in order to determine an understanding of the foundation conditions at the Barkley site. The results are a synthesis of the data with consideration given to the strengths and weaknesses of all the various sources of information.

41. The analysis showed that the foundation could be modeled by three basic foundation units: Unit 1, Unit 2, and Unit 3. The overall foundation thickness is approximately 120 ft. The data showed that each unit had traits which distinguished it from the others. An idealized cross section of the switchyard area showing the spatial relationships between the three foundation units is shown in Figure 27. Foundation Unit 1 consists of clays which typically classify as CL materials. These clays which can be viewed as a topstratum are probably overconsolidated due to desiccation. Generally speaking, Unit 1 is thirty feet thick and lies between the Elevations of 350 and 320 ft. Unit 2 consists of silty sand layers interbedded within a matrix of soft clays (CL). The silty sand layers are generally very thin, on the order of only a few inches thick. These silty sand layers are dirty and on the average contain about 30 percent nonplastic fines. Unit 2 lies between elevation 320 and 305 in the switchyard area and is a little thicker beneath the main dam where it generally lies between Elevations 330 - 295 ft. The boundary between Units 2 and 3 is not flat and is crowned with its peak elevation being between Station 65+00 and 70+00 on the cross section (See Figure 10). Unit 3 is subdivided into three subunits: Units 3a, 3b, and 3c. Unit 3a contains dense sands and gravels which have a relatively high penetration resistance as compared to the sands layers in Unit 2. These sands typically contain less than 15 percent fines and are relatively clean compared to those in Unit 2. Unit 3a lies generally lies between elevation 305 and 295 in the

switchyard area. There are some clay layers present in Unit 3a. Unit 3b, located between elevation 295 and 288, is typified by clays (CL) of low penetration resistance. These clays were detected by nearly all of the CPT's in the switchyard and appear to be a characteristic of this area of the damsite. Unit 3c lies between elevation 288 and bedrock and based on limited amounts of data appear to have characteristics similar to Unit 3a.

42. A quantitative analysis of the CPT data showed that sands make up less than 20 percent of the material of Unit 2. In Unit 3, sand is the dominant material comprising about 60 percent of the total material.

43. Preliminary studies showed that the materials in foundation Unit 2 had a high potential for liquefaction. Thus, in the stratigraphy evaluation it was essential not only to identify and classify the material type but to map its lateral extent. It was not possible to map the extent of individual sand layers from one CPT or SPT sounding to another. However, an excavation in a downstream exposure of Unit 2 revealed that some of these layers were undulating and continuous over fairly long distances in the direction of river flow. This result was carried into the liquefaction studies where the sandy materials in Unit 2 were treated as continuous. The cross section of Figure 27 represents the idealized stratigraphy upon which the seismic response and performance of the dam and foundation were based.

PART III: FINITE ELEMENT ANALYSIS OF TYPICAL SWITCHYARD SECTION AT BARKLEY DAM AND ONE-DIMENSIONAL DYNAMIC RESPONSE ANALYSIS OF TYPICAL SECTION OF THE MAIN EMBANKMENT

General

44. Finite element and 1-D dynamic response analyses were used to evaluate the dynamic responses of representative cross section of both the switchyard and main embankment areas. Static and dynamic finite element analyses were performed on a representative cross-section through the switchyard area, located between Sta 35+00 and Sta 44+00, to evaluate the pre-earthquake static stresses, dynamic response, and liquefaction potential of the embankment and the foundation materials beneath Barkley Dam. The static analysis was necessary since the cyclic strengths (liquefaction resistance) of the foundation and embankment soils are dependent upon pre-earthquake states of stress. The dynamic response analysis was performed to evaluate the response of the dam and foundation to earthquake vibrations and to determine the dynamic shear stresses induced in the soils by the earthquake. A series of 1-D wave propagation analyses were used to approximate the 2-D dynamic response of a typical section of the main embankment (located between Sta 44+00 and Sta 90+00).

45. This part of the report presents the results of the analytical studies. A discussion of the determination of liquefaction resistance is included in Part IV of this report.

Analysis of Switchyard Area

Selection and idealization of representative cross-section finite element and post-earthquake analysis

46. The cross-section used for the finite element analysis is section A-A shown on Figure 27. Preliminary liquefaction and post-earthquake analyses performed on this and other cross sections at Barkley Dam indicated that this was the most critical section in the switchyard area and was an appropriate section to be idealized for the finite element analysis.

Static finite element analysis

47. General: The computer program FEADAM84 developed by Duncan, Seed, Wong, and Ozawa (1984) was used to perform the static analysis of Barkley Dam.

FEADAM84 is a two-dimensional, plane strain, finite element solution developed for calculation of the static stresses, strains, and displacements in earth and rockfill dams and their foundations. The program uses a nonlinear hyperbolic constitutive model developed by Duncan et al. (1980) to estimate the stress history dependent, non-linear stress strain behavior of the soils. Nine parameters are required for the hyperbolic constitutive model. The program is used to perform incremental calculations to simulate the addition of layers of fill placed during construction of an embankment. A description of the constitutive model, procedures for evaluating the parameters, and a database of typical parameter values are given by Duncan et al. (1980).

48. Finite element inputs: FEADAM84 was used to compute the pre-earthquake static states of stress in the embankment. Seven different material types were modeled in the cross-section:

a. Random embankment fill.

b. Compacted embankment fill.

c. Unit 1 - Lean alluvial clay.

d. Units 2 and Unit 3a - Silty sand.

e. Units 2 and 3b - Clay.

f. Unit 3c - Sands and gravels.

g. Submerged compacted embankment fill.

49. The distribution of these materials is shown in Figure 28. Table 3 contains a list of the hyperbolic parameters used in FEADAM84 to model the nonlinear stress strain behavior of each of the materials in the cross-section. Submerged unit weights were used for all materials located beneath the phreatic line. Submerged materials were assigned the same hyperbolic parameters as were their non-submerged counterparts. The hyperbolic parameters were determined by Duncan and Seed (1984) in an earlier study.

50. The finite element mesh developed for the static analysis is shown in Figure 29. This mesh has a total of 265 elements and 293 nodal points. The phreatic line representing the boundary between submerged and nonsubmerged elements is indicated in this figure. This mesh is different than the one used in the dynamic analysis. The mesh used in the dynamic analysis will be discussed in the next section of this part of the report.

51. The dam and its foundation were numerically constructed in two basic stages. In the first stage, the alluvial foundation was "constructed" in ten lifts with each lift being one element high. The computed stresses for

the 221 foundation elements were treated as input to the second stage of the analysis. In the second stage, the 44 elements comprising the embankment were "constructed" upon the preexisting foundation elements in four construction layers, each one element high.

52. Results of the static analysis: The results of the FEADAM84 static stress analysis are presented in the form of contour plots of vertical effective stress, shear stress on horizontal planes, and alpha (ratio of horizontal shear stress to vertical effective stress) in Figure 30 through 32, respectively. The contours were derived from the stresses computed at the centroid of each of the elements in the mesh. The data plotted in Figure 30 show that the contours of vertical effective stress follow the surface geometry of the cross-section. As would be expected, the plot shows that the vertical stresses increase with increasing depth below the ground surface. Additionally, the vertical effective stresses at corresponding depths on the downstream side of the dam are slightly greater than those on the upstream side due to the effect of submergence. The vertical effective stresses are slightly in excess of 10 ksf just above the rock surface at centerline of the dam.

53. Figure 31 shows a contour plot of the initial static shear stresses acting on horizontal planes in the cross section. Due to the sign convention of the program the stresses on the downstream side of the centerline (negative) have the opposite sign of those on the upstream side (positive). The contour of zero shear stress was located near the centerline of the dam (at X = 0). The contours show that the absolute values of shear stress are greatest beneath sloping sections of the surface geometry. It is important to note that beneath the switchyard berm is a zone of relatively low shear stresses, which is a consequence of the relatively flat surface geometry of the berm section.

54. Figure 32 shows contours of the alpha ratio. The alpha values shown represent the ratio of initial static shear stresses acting on horizontal planes to the vertical effective stress. The signs of the alpha values on the downstream side (negative) are opposite those on the upstream side (positive) and reflect the signs of initial shear stresses in these locations. The contours show that the absolute value of alpha ranges from 0 near the centerline to 0.3 on the upstream slope. As was the case with the static shear stresses, the higher alpha values are located immediately beneath sloping sections in the surface geometry on both the upstream and downstream

sides of the centerline. In contrast to this, the alpha values are less than 0.05 in the upstream and downstream free field areas where the ground is level and below the switchyard berm section where the slopes are relatively flat.

55. The pre-earthquake static stress conditions are used in subsequent portions of the seismic stability analysis. They were used in the determination of the cyclic strengths at various locations in the foundation soils, since the cyclic strength at a location is dependent upon the vertical effective stress and alpha value at that point.

Dynamic finite element
analysis of switchyard section

56. General: A two-dimensional dynamic finite element analysis was performed with the computer program FLUSH (Lysmer et al. 1973) to calculate the dynamic response of the idealized cross section to the specified motions. The objectives of this analysis were to determine dynamic shear stresses, peak accelerations at selected points in the cross section, earthquake-induced strain levels, and the fundamental period of the dam at the earthquake-induced strain levels. The information gained from the dynamic analysis is used later as input to the evaluation of liquefaction potential and seismic stability of the idealized cross-section in the event of the design earthquake.

57. Free field ground motions applicable to the Barkley site were input to FLUSH. The free field motions were developed using the computer program SHAKE. SHAKE is a one-dimensional wave propagation code developed at the University of California - Berkeley (Schnabel, Lysmer, and Seed 1972). As is the case with FLUSH, SHAKE uses the equivalent linear model to simulate non-linear soil behavior. A discussion of the processing scheme by which the free field ground motions were developed will be discussed in a following section.

58. Description of FLUSH: FLUSH is a finite element computer program developed at the University of California, Berkeley by Lysmer, Udaka, Tsai, and Seed (1973). The program solves the equations of motion using the complex response technique assuming constant effective stress conditions. Non-linear soil behavior is approximated with an equivalent linear constitutive model which relates shear modulus and damping ratio to the dynamic strain level developed in the material. In FLUSH the differential equations of motions are solved in the frequency domain and an iterative procedure is used to determine the appropriate modulus and damping values to be compatible with the developed

level of strain. Plane strain conditions are assumed in FLUSH. As a two-dimensional, total stress, equivalent linear solution, possible pore water pressure generation and dissipation are not accounted for during the earthquake. Each element in the mesh is assigned properties of unit weight, shear modulus, and strain-dependent modulus degradation and damping ratio curves. FLUSH input parameters for the various zones in the cross section are described later in this part of the report.

59. Description of SHAKE: The one-dimensional computer program SHAKE was used to develop site specific ground motions for input to FLUSH and also to evaluate the dynamic response of the free field at the damsite. SHAKE was developed by Schnabel, Lysmer, and Seed at the University of California, Berkeley (1972). In SHAKE, the wave equation is solved in the frequency domain through the use of the Fast Fourier Transform (FFT). The nonlinear strain dependent soil properties of shear modulus and damping are handled with the equivalent linear procedure, an iterative process which converges upon strain compatible values for modulus and damping. The equivalent linear model is the same constitutive model used in FLUSH. The one-dimensional analysis performed in SHAKE is a total stress analysis. The strain dependent damping and modulus degradation curves for this study are shown in Figure 33.

60. SHAKE is based upon the following assumptions:

a. All layers in the profile are horizontal and of infinite lateral extent. Level ground conditions are assumed to exist, thus prior to the earthquake there are no static shear stresses existing on horizontal planes.

b. Each soil layer in the profile is defined and described by its shear modulus, damping, total density, and thickness.

c. The response of the soil is caused by horizontally polarized shear waves propagating through the soil layers in the system.

d. The acceleration history which excites the soil profile are shear waves.

e. The equivalent linear procedure satisfactorily models the nonlinear strain dependent modulus and damping of the soils in the profile.

61. Finite element mesh design: The mesh used in the dynamic analysis is presented on Figure 34. This mesh has 531 elements and 571 nodes. This mesh is finer than the mesh used in the static analysis which had 265 elements and 293 nodes. The elements were designed to insure that motions in the frequency range of interest propagated through the mesh without being filtered by

the mesh. Using the criterion of Lysmer et al. (1973) the maximum element height was determined using Equation 1:

$$h_{max} = (1/5) \times V_e \times (1/f_c) \tag{1}$$

where

h_{max} = maximum element height

V_e = lowest shear wave velocity compatible with earthquake-induced strain levels in the zone of interest

f_c = highest frequency in the range of interest

62. The low strain amplitude shear wave velocity distribution of the cross section determined from geophysical testing is shown in Figure 35. A cutoff frequency of 6 hz was used for this analysis since embankment dams are typically long period structures. The effective shear wave velocity, V_e, was estimated from a series of one-dimensional calculations using SHAKE to approximate the response of different zones in the embankment and foundation to earthquake shaking. From this information the maximum element heights for the various material zones in the embankment and foundation could be determined by using Equation 1. For example, V_e was estimated to be about 150 fps for a portion of Unit 2 that has a low strain amplitude shear-wave velocity of 700 fps. Hence using Equation 1:

$$h_{max} = (1/5) \times 150 \times (1/6) = 5 \text{ ft}$$

According to Lysmer's criterion the maximum element height should not exceed approximately 5 ft to guarantee that frequencies upto 6 Hz will propagate well through the mesh. The mesh design was completed by performing similar calculations for the other embankment and foundation material zones to determine their maximum element heights.

63. Material properties: The key material properties input to FLUSH were the unit weight and low strain amplitude shear modulus and damping curves for each material type in the cross section. The unit weights, shear-wave velocities, and shear modulus for each of the seven material types in the cross section are shown in Figure 35 and are listed in Table 4. The modulus degradation and damping curves used in this study for both one- and

two-dimensional dynamic response analyses are shown in Figure 33. These curves were developed by Seed, et al. (1984) for cohesionless soils.

64. Site specific ground motions: In Volume 2 of this series Krinitsky (1986) specified peak ground motion parameters for a firm soil site near Barkley Dam. The design event is a magnitude $m_b = 7.5$ based on a repeat of the New Madrid events of 1811-12. Krinitsky (1986) recommended using the Santa Barbara Courthouse Record, S48°E component, from the Kern County, CA earthquake of 21 July 1952 as the design accelerogram. This record was to be scaled to 0.24 g and applied to the ground surface of the firm soil site. After scaling, this accelerogram had a peak velocity of 35 cm/sec and a duration (>= 0.05 g) of about 60 sec. Figures 36 and 37 show the acceleration history of the scaled Santa Barbara record and its response spectra, respectively. These motions represent the response that would be measured at the surface of a firm soil site in the vicinity of Barkley dam. The periods of interest for this study range from about 0.5 to 2 sec. Examination of the response spectra in Figure 37 shows that these periods occur in the portion of the spectra with the strongest response, thereby indicating that the Santa Barbara record was sufficiently rich in the frequencies of interest and would be appropriate for the dynamic response calculations. The earthquake record and scaling specifications were approved by the Technical Advisors (Drs. H. B. Seed, G. Castro, L. T. Long, and A. Nieto) on 27 August 1980. (See Appendix A to Volume 1).

65. The site specific input to the FLUSH analysis were developed using the a process illustrated in Figure 38. The discussion which follows is keyed to numbers which are tied to locations on the schematic. A firm soil profile and a deconvolution procedure using SHAKE were employed to bring the motions to the site's free field soil profile. The firm soil profile (left hand side of Figure 38) is shown in Figure 39. The profile used in this analysis was developed from data available from the soils beneath the Santa Barbara recording station. The scaled Santa Barbara Courthouse record was input to SHAKE at the ground surface of this profile. These motions were propagated through the profile for the purpose of obtaining the acceleration history at the rock outcrop location (Point 3). The peak acceleration at baserock (Point 2) of the firm soil profile was 0.18 g and the peak acceleration at the rock outcrop position (Point 3) was 0.18 g. A comparison of the peak accelerations at the ground surface (0.24 g) and at base rock indicates that the baserock motions

were amplified as they propagated through the firm soil profile. The acceleration history at the rock outcrop location, Point 3, and its response spectra for 5 percent damping level are presented in Figures 40 and 41, respectively.

66. The evaluation of the dynamic response of the free field soil profile at Barkley dam was the next step in the development of the ground motions input to FLUSH. The free field soil profile used in the analysis is presented in Figure 42. Figure 42 shows that the idealized free field profile has a height of 90 ft and was subdivided into 13 soil layers for analysis. As documented in Volume 3, the 90 ft height of this soil profile was determined from borings used for geophysical testing located downstream of the switchyard area. Figure 42 shows the total velocities and low-strain amplitude densities assigned to each of the layers in the profile. These properties were determined based on information obtained from the geophysical and soil exploration programs. The dynamic response analysis was performed by exciting the profile with the rock outcrop accelerogram (Point 3) shown previously in Figure 40. Key information sought from the response were the earthquake-induced peak accelerations, dynamic shear stresses in each layer, and the acceleration history at the ground surface of the profile. Figure 43 is a plot of the peak accelerations as a function of depth for the profile. The plot shows that the general trend is for the peak accelerations to decrease with depth indicating that the soil profile amplifies the baserock motions. The baserock motions (Point 4 on Figure 38) were 0.16 g and the motions at ground surface (Point 5) were 0.26 g. The peak acceleration at the outcrop location (Point No. 3) is slightly higher than that of the baserock location (Point No. 3) because the outcrop location represents a free surface where the energy from incident seismic waves is totally reflected. On the other hand, at baserock a portion of the energy is transmitted across soil-rock interface, and the remainder is reflected.

67. Figure 44 is a plot of the effective dynamic shear stresses induced by the earthquake. The effective stresses are 65 percent of the value of the peak shear stresses. Figure 44 shows that the dynamic stresses increase with depth.

68. The ground surface acceleration history (Point 5) is presented in Figure 45. These motions were subsequently used as input to the FLUSH analysis. The response spectra for this accelerogram is shown in Figure 46.

69. The peak accelerations at the five key locations used in ground motion development process are shown in Figure 47. A comparison of the peak acceleration value at Point 1 to that at Point 5 shows that there is only a slight increase in value from the point where the ground motions were introduced to the target location in the system. A plot showing the ratio of spectral acceleration at Point 5 to those at Point 1 is shown on Figure 48. This plot indicates which frequencies propagate best through the two profile system. This plot shows that the ground motions developed in the system of Figure 38 tends to amplify the spectral acceleration values for periods above 0.6 sec. The greatest spectral amplification occurs at a period of about 1.5 sec. The short period spectral values below 0.6 sec. are either deamplified or amplified only slightly.

70. Results of the FLUSH analysis: The dynamic response of the representative cross section to the site specific input motions was computed with FLUSH. The cross section was excited by applying the accelerogram shown in Figure 45 to a control point located at the ground surface of the site's free field profile. FLUSH duplicates the free field response computations performed with SHAKE. The motions of the free field response were transmitted to the finite element mesh across transmitting boundaries which separate the free field profile from the finite element mesh (See Figure 38). The dynamic response of each element and nodal point in the finite element mesh to these input motions were computed with FLUSH. From these calculations the maximum earthquake-induced cyclic shear stress computed for each element over the entire duration of shaking was determined. The maximum value was multiplied by 0.65 to determine the average cyclic shear stress imposed by this earthquake (Seed and Arango, 1983). Contours of the average earthquake-induced dynamic shear stress caused by the input accelerogram are shown in Figure 49. The plot shows that the contours are approximately parallel to the dam's surface geometry. The shear stresses are zero at the ground surface and increase with depth. The dynamic stresses are highest just above base rock elevation where the 1250 psf contour runs from the upstream side of the mesh to a location about 350 ft downstream of the centerline. Safety factors against liquefaction in the foundation elements were later calculated using the dynamic shear stresses presented in this plot.

71. The peak acceleration for each nodal point in the finite element mesh were also computed with FLUSH. The peak accelerations from selected

nodal points are presented in Figure 50. The data in the figure show that there is a general trend for the peak acceleration to decrease with depth below the ground surface. The maximum peak acceleration occurred at the crest of the dam where the value was 0.28 g.

72. The effective strain levels induced by the Santa Barbara Courthouse record in the FLUSH analysis are shown in Figure 51. The cross hatched area indicates the zone of elements which had the largest earthquake effective cyclic shear strains in the FLUSH analysis. The effective strains in these elements ranged from 0.7 to 1 percent. This area coincides largely with the foundation soils of Units 2 (interbedded sands within clay) and 3b (weak clay layer). This area extends completely across the section from the downstream in the vicinity of the switchyard to the upstream side of the dam. The modulus degradation curves in Figure 33 shows that for the these levels of strain the modulus would degrade to about 8 percent of its maximum value. This level of strain is consistent with that estimated in the mesh design for this region of the foundation and corresponds to a cutoff frequency of about 6 hz. Figure 51 also shows that the strain levels for elements in the embankment and foundation Units 1 and 3c would have effective strains ranging between 0.06 and 0.2 percent. The strain levels in these areas are approximately one-half to one order of magnitude lower than those for the cross hatched area.

73. The lengthening of the embankment fundamental period during earthquake shaking is another measure of strain softening in the embankment materials. The pre-earthquake period of the embankment and foundation system were estimated using a simplified procedure developed by Sarma (1979). FLUSH was used to estimate the period of the embankment and its foundation at the earthquake induced strain levels. The calculations shown in Figure 52 show that the pre-earthquake fundamental period of Barkley Dam was 0.75 sec. The period at the earthquake induced strain levels was determined to be 1.75 sec. A comparison of the two values indicates that the period lengthens by a factor greater than two as a result of the straining caused by the earthquake. This result is consistent with the results of the FLUSH analysis which showed a significant amount of softening was occurring in Units 2 and 3b due to the large strains induced by the earthquakes. A comparison of the pre-earthquake and effective fundamental periods of the dam and foundations system with the response spectra of the input ground motions from Point 5 (of Figure 46) is

shown in Figure 53. This comparison shows that both periods closely match the periods of the strongest part of the response spectra. This indicates that the Santa Barbara Courthouse accelerogram is rich in frequencies having periods which lie between the pre-earthquake and effective fundamental periods of the system.

Analysis of Main Embankment

General

74. The dynamic response of the main embankment was modelled using a series of 1-D wave propagation calculations to approximate a 2-D dynamic response. The 1-D analyses were performed using SHAKE which were described earlier in this report. The principal objective of these analyses were to evaluate the earthquake induced shear stresses in the foundation units beneath the embankment. The section selected to represent the main embankment was Sta 64+00 (location shown in Figure 4). This section was selected because it is near locations where soil sampling and geophysical testing were performed (See Volume 3).

One-dimensional soil profiles

75. Four 1-D soil profiles were developed to approximate the dynamic response of the representative cross section. The locations of these profiles are shown in Figure 54. Detailed information concerning the sublayering and total densities, and the shear-wave velocities used for each of the four profiles are presented on Figure 55 through 58. The shear-wave velocities in Units 2 and 3 of Profiles 2, 3, and 4 were adjusted to account for the increased overburden stresses caused by the overlying embankment.

Results of SHAKE analyses

76. Each of the four profiles was excited by the acceleration history shown on Figure 40 which corresponds to the rock outcrop location of Point No. 3. As discussed previously, the peak acceleration at this point was 0.18 g. Peak accelerations and dynamic shear stresses (65 percent of the peak value) were the key pieces of information sought from the dynamic responses for each profile.

77. The plots of peak acceleration versus depth for Profile 1 through 4 are shown on Figure 59 through 62, respectively. For each of the four responses, the peak accelerations showed a general tendency to increase from

the baserock level. The amplification ratios from of the peak acceleration at the outcrop to the peak acceleration at the ground surface varied from 1.57 to 1.25 for Profiles 1 through 4.

78. The dynamic shear stresses induced by the input accelerogram are plotted in Figures 63 through 66 for Profiles 1 through 4, respectively. The effective stresses plotted in the figures represent 65 percent of the peak dynamic shear stress for each layer in the profile. In each of the responses the effective stresses increased with depth. In Part IV the liquefaction potential of the foundation soils are evaluated by comparing the cyclic strengths with the dynamic stresses shown in Figures 63 through 66.

PART IV: EVALUATION OF LIQUEFACTION POTENTIAL OF THE FOUNDATION OF BARKLEY DAM

General

79. The liquefaction potential of the soils in the three foundation units beneath the switchyard and main embankment areas was evaluated with Seed's performance-based approach (Seed et al. 1983, 1984). The liquefaction potential is quantified by computation of safety factors against liquefaction (FS_L) which compare the cyclic strength of the soil with the dynamic stress induced by the earthquake. Seed's approach uses Standard Penetration Test (SPT) blowcounts to determine the cyclic strength (liquefaction resistance) of a soil deposit. The SPT blowcounts are corrected to a confining stress of 1 tsf and an energy level of 60 percent. The dynamic stresses were determined from the finite element analysis documented in Part IV of this report. The Seed procedures were developed for evaluating the liquefaction potential of sand, silts, silty sands, and gravels, however, screening criteria are included for distinguishing liquefiable clays from nonliquefiable clays.

80. For this study, liquefaction is defined as that condition which is reached in a soil deposit when earthquake induced cyclic stresses of sufficient magnitude, sustained for a sufficient number of cycles, cause the deposit to reach strain levels large enough to increase pore pressures and reach the residual strength of the soil.

81. In this study, the Seed approach was used to identify and estimate the extent of the soil deposits in the foundation most likely to liquefy as a consequence of the design earthquake ground motions. Additionally, the Seed method was extended to estimate the earthquake induced pore pressure buildup in areas of the foundation where safety factors against liquefaction are greater than unity. Zones showing the extent of liquefied material were plotted on the idealized cross section. Information from the liquefaction analysis was used to estimate the post-earthquake strengths.

82. The cyclic strengths estimated from the Seed approach depend primarily upon the SPT blowcounts and the fines content of the material. The application of this approach is complicated by the fact that the alluvial foundation consists of many thin sand soil layers with varying amounts of fines. In sandy deposits of this sort it is difficult to estimate the "true"

SPT penetration resistance of these materials. The CPT with its abilities to provide great detail and a continuous record of information was used to overcome some of these difficulties. Procedures were developed to convert the CPT penetration resistances into their equivalent SPT blowcounts so that Seed's performance-based charts could be used to estimate the cyclic strength.

SPT Blowcounts

83. As described in Volume 3 of this series, 44 Standard Penetration Test and eleven undisturbed borings were made to investigate the nature of the three units in the alluvial foundation. A variety of procedures were used to conduct the SPT borings: for some holes rope and pulley systems were used, and in other holes trip hammer equipment was used. In some holes the SPT spoon was driven continuously with no clean-out distance between drives, and in other holes the clean-out distance was at least one foot as recommended for standard driving procedures (ASTM, 1991). Since the split spoon typically crossed several layers over the 18-in. drive, several jar samples were taken from each split spoon to identify the layers involved in the drives. All information pertaining to the driving procedure and the laboratory index tests from the jar samples were stored in a large database for analysis of the Standard Penetration Test data from the site.

84. Sands, silts, silty sands, and gravels are material types which in the past have been known to liquefy as a result of earthquake shaking. Thus, in order to evaluate the cyclic strength and subsequent liquefaction potential of the sand layers, particularly those in Unit 2, it was necessary to determine the SPT blowcounts of the sand layers present in the foundation.

Data reduction procedures

85. The measured blowcounts were corrected to determine the value that would be obtained had the Standard Penetration Test been performed under a specified set of standard conditions. Blowcounts applicable to standard conditions can be compared directly on a one to one basis. Factors which have a major influence on the measured SPT blowcounts include: the energy efficiency of the drilling procedure, the effective vertical stress, and the fines content of the soil. According to the Seed procedure, the standard conditions to which all blowcounts are reduced include: a vertical effective stress of 1 tsf, 60 percent energy efficiency, and a fines content of less than or equal

to 5 percent. Blowcounts reduced to the standard conditions are designated N_{1c}. The following paragraphs discuss the data reduction procedures employed to determine the blowcounts for standard conditions.

86. Overburden correction: The field measured blowcounts were corrected to a vertical effective stress of 1 tsf by the factor C_N shown in Figure 67. The curve on this plot shows that C_N is a function of the overburden stress. The vertical effective stresses were determined in the static finite element analysis described in Part III of this report. The effective stress correction factor C_N can be expressed in an equation and is approximately equal to the vertical effective stress in tsf raised to the power of n, where n = -0.55 for a relative density of approximately 50 percent and -0.45 for a relative density of approximately 70 percent. An exponent of -0.5 was used in this study which is generally adequate for most calculations. The equations used for the overburden corrections are shown below:

$$N_1 = N_{meas} \times C_N \tag{2}$$

$$C_N = (\sigma_v')^n \tag{3}$$

where

N_1 = SPT blowcount corrected to an effective stress of 1 tsf

N_{meas} = field-measured SPT blowcount

C_N = overburden correction factor

σ_v' = vertical effective stress (TSF) from static finite element analysis

n = -0.55 for a relative density of about 50 percent

= -0.45 for a relative density of about 70 percent

87. Energy delivery: The field measured blowcounts were corrected to an energy delivery of 60 percent efficiency in accordance with the guidelines described by Seed, Tokimatsu, Harder, and Chung, (1984). This energy correction accounts for the efficiencies of the different types of equipment used in performance of the Standard Penetration Test in the field. No correction was made to account for continuous Standard Penetration Testing, that is, testing with no clean-out space between 18-in. drives. Upon comparison of

continuously measured SPT's and standard SPT's with a clean-out depth of at least 1 ft, no corrections seem to be warranted. In the manner described above and in the preceding paragraph, the field measured SPT blowcounts were corrected to a confining stress of 1 tsf and to an energy efficiency of 60 percent. These corrected blowcounts, designated as $(N_1)_{60}$, along with knowledge of the fines content of the soil were used to enter the cyclic strength charts for liquefaction evaluation shown in Figure 68. The cyclic strengths pulled off the chart apply to Magnitude 7.5 events.

88. Adjustment for fines content: An additional adjustment was made to each blowcount, $(N_1)_{60}$ to correct for the effect of the fines content (percentage passing the No. 200 sieve). The corrected blowcount is termed the equivalent sand blowcount and designated as N_{1c}. The subscript c indicates that corrections for overburden pressure, energy efficiency, and fines content have been performed on the field value of blowcount. The equivalent sand blowcount, N_{1c}, has the same cyclic strength as the $(N_1)_{60}$. For example, the N_{1c} value is 15 blows/ft for a material having a $(N_1)_{60}$ value of 12 blows/ft and a fines content of 15 percent as illustrated in Figure 68. The determination of N_{1c} for all SPT blowcounts allows SPT data obtained in materials having different fines contents to be compared on a one-to-one basis.

89. Many samples retrieved with the SPT sampler crossed several sand layers during the 18 in. drive. The fines content of SPT's performed in predominately sandy materials represented a weighted average of the fines contents of each of the thin sand layers retrieved with the split spoon. The fines contents of each of these sand layers was determined by dividing the 18 in. sample into smaller samples. The equivalent clean sand blowcount was then determined from the $(N_1)_{60}$ value and the observed fines contents.

Results of SPT Data Analysis

90. Analysis of the SPT data was complicated by the complexities of the three foundation units at the damsite, particularly those of Unit 2. Meaningful results could only be obtained by keeping the findings of the stratigraphy evaluation in mind during the analysis of the data. Thus, the data were analyzed with the view that the interbedded sands and clays were distinguishable components and that the characteristics of each should be examined separately.

91. The SPT data analysis of the sandy portion of the foundation was performed for the purpose of determining the blowcounts $(N_1)_{60}$ and fines content. The cyclic strength of the foundation sands could then be determined with these parameters from Seed's charts.

92. The SPT data analysis of the clayey samples of the foundation was performed to assess their potential for liquefaction. Clayey soils having the potential for liquefaction were identified using empirically developed criteria recommended by Seed (1983) based on the findings of Wen-Shao (1981). These are commonly referred to as the "Chinese Criteria".

93. The entire SPT database was scanned boring-by-boring and the data were evaluated according to a set of criteria designed to establish the liquefaction potential of the materials in the foundation, particularly Unit 2. These criteria were selected to account for soil classification (sand or clay), soil losses from the bottom of the sample, blow counts (if sandy), and the identification of nonliquefiable clay samples. The criteria used in the data analysis are listed on Table 5. The analysis of the SPT data was performed on a hole by hole basis. An example of the results is shown on Table 6 for Boring BEQ-30. The tabulations for the other SPT borings are listed in Appendix C. Descriptions of each of the headings for each column of data in Table 6 are listed in Table 7. The tabular printout shows three basic types of data: soil identification data for each data point, laboratory test results (grain size and index data) and measured and corrected blowcounts. Details on how the information contained in these tables was used to evaluate the characteristics of the sands and clays in the foundation are discussed in the following sections.

Evaluation of liquefaction potential for clayey soils

94. As stated previously, the liquefaction potential of the clays were evaluated using the Chinese criteria. These criteria are based upon evidence of field observations which indicated that clayey soils also have the potential for producing ground surface expressions of liquefaction in the form of sand boils. Numerous observations of liquefaction were made in China, due to earthquakes that occurred in the 1970's, in particular, the Tangshan earthquake of 1976. Based on studies of these observations of liquefaction reported by Wen-Shao (1981), Seed, Idriss and Arango (1983) proposed the following criteria for identification of a potentially liquefiable clayey soil:

a. Liquid limit < 35 percent.

b. Natural water content > 0.9 times the liquid limit.

c. Percent finer than 0.005 mm < 15 percent.

In the analysis of the soils at the Barkley site, it was assumed that fine-grained foundation soils meeting all three of these criteria would liquefy as a result of the design earthquake.

95. The clayey samples were distinguished from the sandy samples based on the first letter of the assigned soil identification listed in the tabular printouts in the fifth column of the SPT data base tables (see Table 6 and Appendix C). The first letter identified the soil type of the major component for that particular sample and the second letter (if necessary) was a descriptor. For example, if the identification was "CS" then the sample consisted mainly of clays with some sand present. Thus, the samples were considered clayey soils if "C" was the first letter. These identifications were developed specifically for this project and are not to be confused with Unified Soil Classification System classifications which may have similar designations.

96. Plots of the liquid limits versus plasticity index for the laboratory samples from Units 2 and 3 which were identified as clays are shown in Figures 69 and 70, respectively. Unit 2 soils were considered to be those soils lying between elevations 320 and 305 ft and Unit 3 soils were considered those located below elevation 305. The figures show that the clay soils from both units plot above the "A-Line" and classify as CL according to the Unified Soil Classification System. Also the data from both Units 2 and 3 fall within the same range indicating that Units 2 and 3 are very similar. The data shows that the liquid limit of most of the samples from Units 2 and 3 is less than 35 percent which meets the first (a) of the three Chinese criteria for being considered liquefiable. The average liquid limit of these samples was about 30 percent.

97. Histograms of the ratio of water content to liquid limit (w_n/LL) were used to evaluate the second criteria which states that the ratio must be greater than 0.9 for the clay to be considered liquefiable. The w_n/LL histograms for Units 2 and 3 are shown in Figure 71 and 72, respectively. As was the case with the PI vs LL plots these histograms show the w_n/LL distributions for Units 2 and 3 are very similar and fall within the same range. The mean w_n/LL ratio is about 0.80 for Unit 2 and 0.85 for Unit 3 which shows that the

clays in these units have high water contents. The w_n/LL ratios are only marginally lower than that given by criteria (b) for being considered a liquefiable soil.

98. The third criteria involved determining whether the percentage passing the 0.005 mm of the clays was typically less than 15 percent. This criteria is checked in the form of charts which show w_n/LL ratio plotted against the percentage of material finer than 0.005. The charts for Units 2 and 3 are shown in Figure 73 and 74, respectively. Since it has been established that the average in situ liquid limit is less than 35 percent, any point falling within the cross hatched boxes on these figures would classify as a liquefiable clay which meets all three of the "Chinese criteria". The plots show that the percentage passing 0.005 mm fall within the same range for both Units 2 and 3 which indicates the "clays" within these two zones are similar. The plots also show that nearly all the data points fall outside the shaded region; this indicates the clays are not liquefiable.

99. In summary, analysis of SPT data from clayey soils shows that the clayey soils of Unit 2 and 3 are very similar having liquid limits, w_n/LL ratios, and percentages passing 0.005 mm size which fall within the same ranges. Additionally, the clays in these units were considered to be nonliquefiable since all three of the Chinese criteria are not met. The major factor in arriving at this conclusion is the fact that SPT samples show that almost invariably more than 15 percent of these clay materials are finer than 0.005 mm. Additionally, the CPT data showed that the soils of Unit 3b were predominately clayey in nature; thus, in the remainder of the analysis these were treated nonliquefiable clays.

Analysis of SPT blowcounts and fines contents of sandy soils

100. As mentioned previously, since the Seed approach was adopted for use in this study the blowcounts and fines contents are required pieces of information in the determination of the cyclic strengths of sandy soils. The SPT database of Volume 3 was analyzed to determine these parameters for the predominately sandy samples. In this analysis the measured blowcounts and fines contents were determined for each sand sample and were corrected using the procedures discussed earlier to yield values for the corrected blowcounts $(N_1)_{60}$ and ultimately N_{1c}.

101. A statistical analysis was performed on the N_{1c}'s so that the SPT data in different areas of the foundation could be compared directly. Since a high degree of uncertainty is introduced into the interpretation of SPT drives through multiple layers of fine-grained soils and soil mixtures, the SPT database was searched to separate out the more reliable SPT data obtained in the sandier materials. Sandy materials were considered to be those drives where 0 to 50 percent passed the No. 200 sieve. Figure 75 shows a histogram of the equivalent sand blowcounts in Unit 2 between elevations 305 and 320 ft in the switchyard area. The switchyard area encompasses the area between Sta 33+50 and 44+00. The mean value and standard deviation are 13.6 and 3.0 blows/ft for this set of data. The histogram was constructed from only 22 samples.

102. In the main embankment area, analyses were performed to determine the statistical distributions of $(N_1)_{60}$ blowcounts, percentage passing the No. 200 sieve (fines content), and the N_{1c} blowcounts. The main embankment encompasses the area between Sta 44+00 and 90+00. Histograms showing the statistical distributions of $(N_1)_{60}$ and fines content N_{1c} are presented on Figures 76 and 77, respectively. Statistical analysis indicates that the distribution of $(N_1)_{60}$ has a mean value of 15.82 blows/ft and a standard deviation of 8.30 blow/ft. The fines content has a mean value of 23.2 percent and a standard deviation of 16.2 percent. A histogram showing the distribution of equivalent sand blowcounts, N_{1c}, for the sandy soils of Unit 2 for the main embankment is shown in Figure 78. The mean and standard deviation for this set of data was 20.7 and 6.5 blows/ft based on 83 samples.

103. Histograms similar to those of Unit 2 of $(N_1)_{60}$, fines content, and N_{1c} were developed from SPT data obtained in Unit 3 (below Elev 295). Figure 79 shows the $(N_1)_{60}$ distribution which has a mean value of 23.8 blows/ft and a standard deviation of 8.9 blows/ft. The fines content data of Figure 80 had a mean value of 16.1 percent and a standard deviation of 10.6 percent. The histogram showing the distribution of N_{1c} or the sands in Unit 3 is shown on Figure 81. The data in Figure 81 was obtained from SPT borings located across the entire site which includes the switchyard and main embankment areas while the data in Figures 76, 77, 78, 79, and 80 were obtained from borings located only in the main embankment area. The mean and standard deviation were 24.3 and 9.9 blows/ft based on 258 samples. The statistical analyses of the SPT data for $(N_1)_{60}$ and fines content of the main embankment areas for Units 2 and 3 are summarized in Table 8. The statistical

analysis of the data for the equivalent clean sand blowcounts, N_{1c}, obtained from the SPT are summarized in Table 9.

104. The following conclusions were drawn from analysis of the data presented in Figures 75 through 81:

a. The Unit 2 equivalent sand blowcounts, N_{1c}, in the switchyard area blowcount generally are lower than those of the main embankment area.

b. The equivalent sand blowcounts, N_{1c}, in Unit 3 are higher than the Unit 2 blowcounts for both the switchyard and main embankment areas.

There are several possible explanations for the above results. Conclusion (a) suggests that the Unit 2 sand layers of main embankment area are either denser and/or thicker than those of the switchyard area. The small number of samples (23) obtained in switchyard area of Unit 2 provides further evidence of this since it is more difficult to find predominately sandy samples if the layers are thinner based on the rationale that thinner layers are more difficult to sample. Additionally, the blowcounts in these thin sand layers probably do not reflect the true penetration resistance of these sands since they are influenced by the soft clays which dominate Unit 2. Better estimates of the equivalent sand blowcounts were observed in Unit 2 of the main embankment area (83) and Unit 3 where a far greater number of samples were recovered (248). The results indicate that the sands obtained here were thick enough allow the true penetration resistance in these areas to be approximated with the SPT.

105. The low blowcounts measured with the SPT in Unit 2 of the switchyard area suggested that this section of the foundation would be the most critical in terms of the seismic stability of the embankment. However, the cyclic strength of the sands in this area were difficult to evaluate due to the problems associated with the sampling and the performance of the SPT in thin sand layers embedded in the soft clays. Hence, the Cone Penetration Test (CPT) was used to obtain more and higher quality data with regard to the penetration resistances of the thin sand layers. This data was used to better evaluate the cyclic strength of the thin sand layers as will be discussed in the next section.

Prediction of SPT Blowcounts from Cone Penetration Test Data

CPT prediction of $(N_1)_{60}$

106. The CPT prediction of the Standard Penetration Test (SPT) blow count, N, is an important requirement for the evaluation of the liquefaction potential of Unit 2. Seed's empirically-based liquefaction analysis (Seed, 1979; Seed et al. 1983; Seed et al. 1984;, and Seed, 1986) was developed from large amounts SPT data and observations of liquefaction in the field. During the period of this study an adequate independent database relating CPT values to liquefaction susceptibility did not exist and the most advanced method for SPT $(N_1)_{60}$ prediction from CPT data was that developed by Olsen (1984) and Olsen and Farr (1986). The $(N_1)_{60}$ prediction chart used in this study is shown in Figure 82. This method uses the results of both the tip resistance (q_c) and sleeve friction (f_s) measurements from the CPT, and is sensitive to the soil type and consistency.

107. For the last 20 years, a q_c/N ratio of 4 to 5 was used to estimate the SPT blowcount in sand (Robertson et al. 1983). A widely used q_c/N technique for SPT prediction requires an estimate of D_{50} from a nearby boring to determine the q_c/N ratio from which the SPT N value is calculated (See Figure 83 taken from Robertson, Campenella, and Wightman 1983). Additional research (Schmertmann, 1976, 1979a, and 1979b; Douglas, Olsen, and Martin, 1981) has shown that the SPT side friction can be a major contributing factor for loose and overconsolidated soil conditions. Therefore, the CPT sleeve resistance can make an important contribution in some sands. The side friction and end-bearing resistance components of stress acting on the SPT sampler during driving can be modeled with the CPT probe as shown in Figure 84. Contours of q_c/N_1 shown on the Soil Characterization-N_1 Prediction Chart (Figure 82) are not parallel to the CPT soil characterization lines (i.e. soil type lines), which indicates that the use of the D_{50} to q_c/N correlation may lead to erroneous results. Figure 82 does show that a clean sand in the normal consistency zone of the CPT soil characterization chart has a q_{cn}/N_1 of 4 to 4.5 which equals that determined from Figure 83 for a D_{50} of 0.25 mm. Since the q_c and N require different confining pressure correction factors, the ratio q_{cn}/N_1 more accurately predicts SPT blowcounts, especially at high confining stress levels. Olsen's technique was used to interpret the CPT data

from the Barkley site because of the perceived advantages described above, particularly because the me.... is sensitive to soil type and consistency.

CPT prediction of fines
corrected blowcounts, Nlc

108. The CPT prediction of the Standard Penetration Test (SPT) blowcount, N, is an important factor for liquefaction evaluation. Prediction of N_{1c} allows not only clean sands but dirty sands and sandy mixtures to be evaluated for liquefaction resistance.

109. In the SPT liquefaction evaluation procedure, the percent passing the No. 200 sieve is used to determine the increase required in N_1 to correct for the fines content to determine the equivalent sand value of N_{1c}. CPT Soil Characterization Numbers (SCN) can be matched to the percent passing the No. 200 sieve (Douglas and Olsen, 1981) in order to calculate the blowcount corrected for fines, N_{1c}. Figure 85 shows estimated SCN lines which correspond to 5 and 35 percent passing the No. 200 sieve. These lines also correspond to soils having mean grain sizes of 0.25 mm and 0.15 mm.

110. Also shown in this figure are the fines corrections to be added to the measured blowcounts to determine N_{1c} based on the D_{50} and percent passing the No. 200 sieve when the fines correction was determined. The CPT predicted fines correction is somewhat higher for clayey and mixture materials (e.g. clayey sands) than the SPT approaches because the CPT approach distinguishes (to some extent) between silty fines and clayey fines, whereas the SPT method has the same correction for both silty and clayey fines. The CPT fines correction is based on the trends from cyclic laboratory tests as well as SPT trends for sands and dirty sands. The combination of this data indicates an increased fines correction to determine N_{1c} for soil mixtures with more clayey than silty fines.

111. As shown in Figure 86, the fines-corrected equivalent sand blowcount, N_{1c}, can be determined by adding the appropriate fines-content correction, ΔN_1, to the predicted blowcount. The fines corrections from Figure 85 were combined with the predicted blowcounts (values that would be measured in the field), corrected to 1 tsf, N_1, from Figure 82, to obtain the overburden pressure-corrected, fines-corrected blowcounts, N_{1c}, shown in Figure 87.

112. In situ, a SPT measurement requires a drive of 1.5 ft (ASTM 1991). At layered sites, use of the CPT to predict SPT-N values has some advantages over actual SPT drives because the CPT is more precise for measuring the prop-

erties of thin layers. A commonly used rule of thumb suggests that a soil layer should have a thickness of between 10 and 20 times the diameter of the cone before the "true" penetration resistance can be observed (Robertson and Campanella 1986). Thus, in the case of a standard cone with a diameter of 1.4 in. the rule of thumb suggests the minimum layer thickness for full tip resistance should be between 14 and 28 in. to reliably correlate CPT data to engineering properties. However, for this study it was hypothesized that due to compensating errors in soil classification and fines corrections, the CPT could predict N_{1c} for soil layers less than 1 ft thick. The reasoning behind these compensating errors is described in the following paragraphs.

113. For the Barkley project the majority of the thin sand lenses of concern are on the order of 6 to 12 inches thick. Figure 88 illustrates how both the tip resistance and friction sleeve readings are influenced in a thin sand layer surrounded by thicker layers of softer clay. When the probe is in the sand layer, the tip resistances are influenced by the softer underlying clay which is yielding ahead of the probe. This behavior causes the measured tip resistance to be lower than the "true" tip resistance. This results in the N_{1c} blowcount being underestimated after the CPT to SPT correlations are applied. Conversely, since the thickness of the sand layer is not all that much different than the 4-inch length of the friction sleeve, the clay layers contribute to the friction sleeve values observed while the probe is in the sand. This situation leads to an overestimation of the fines content of the sands and an error in the CPT soil classification. This results in the N_{1c} blowcount being overestimated after the correlations are applied.

114. The errors described in the preceding paragraph are compensating. However, it was felt that error in the fines content only partially compensated for the error in the observed tip resistances. Hence, the equivalent N_{1c} blowcount predicted by the correlations was less than the "true" value. As a result, the cyclic strengths were increased by a factor to account for the thin sand layers present in Unit 2. This factor, K_{layer}, is further described later in this part of the report.

CPT predicted
blowcounts - Unit 2

115. A statistical analysis was performed on CPT predicted $(N_1)_{60}$ and N_{1c} blowcounts to determine the appropriate values to use for the determination of cyclic strength for the sands in Unit 2. The CPT predicted blowcounts

for $(N_1)_{60}$ and N_{1c} for sandy soils were estimated from the tip and sleeve resistances obtained from the Cone Penetration Tests using Olsen's charts in Figures 82 and 87, respectively. Sandy soils were those whose Soil Characterization Numbers (SCN) were greater than 2 (See Figure 18). In querying the CPT database, Unit 2 was assumed to lie between Elevations 320 and 305. A histogram showing the distribution of $(N_1)_{60}$ is shown in Figure 89. This distribution includes 1119 data points, has a mean value of 14.5 blows/ft and a standard deviation of 8.9 blows/ft. This distribution is skewed to the right. The histogram showing the distribution of CPT predicted N_{1c} is shown in Figure 90, determined from the same 1119 basic CPT data points as the $(N_1)_{60}$ histogram, has a mean value of 19.0 blows/ft and standard deviation of 7.2 blows/ft. This histogram shows that CPT predicted N_{1c} has nearly a symmetrical triangular distribution. The CPT predicted average fines content (percent passing the No. 200 sieve) is estimated to be about 15 percent from the mean values of N_{1c} and $(N_1)_{60}$ and use of Seed's cyclic strength chart in Figure 68. This compares with the fines content of 27.7 percent estimated from 160 samples obtained from site-wide SPT borings in both the switchyard and main embankment areas whose distribution is shown in Figure 13.

116. A summary of the statistical analysis is listed on Table 10. It is important to point out that the measured SPT N_{1c} blowcount distribution in Figure 75 was 13.6 blows/ft (based on only 22 samples) which is about 28 percent lower than the 19.0 blows/ft estimated for the CPT predicted N_{1c}. The difference in average N_{1c} determined with the two test techniques is due to the problems associated with estimation of properties of the thin sand layers. The ability of CPT to predict SPT blowcounts is demonstrated in the statistical analysis of the data obtained in the thicker and denser Unit 3 sands which is discussed in the following section.

CPT predicted
<u>blowcounts - Unit 3</u>

117. In like manner to that of Unit 2 the CPT was used to predict the SPT $(N_1)_{60}$ and N_{1c} blowcounts for Unit 3 sands. In the statistical analysis of the CPT data, the Unit 3 sands had SCN's of two or greater and were assumed to be situated below Elevation 305. The histogram for CPT predicted $(N_1)_{60}$ is shown in Figure 91. The distribution, from 7037 combinations of CPT tip and sleeve readings, has a mean of 24.3 blows/ft and a standard deviation of 8.9 blows/ft. The histogram of CPT predicted equivalent sand blowcounts, N_{1c},

is shown in Figure 92. This distribution derived from the same 7037 samples distribution has a mean of 25.7 blows/ft and a standard deviation of 7.6 blows/ft. The relatively large number of data points in Unit 3 (as compared to Unit 2) indicates that sands constitute a greater percentage of Unit 3 than in Unit 2. The CPT predicted fines content (percent passing the No. 200 sieve) is estimated to be about 7 percent from the mean values of N_{1c} and $(N_1)_{60}$ and use of Seed's cyclic strength chart in Figure 68. This result compares with the mean of 15 percent and the mode of 6 percent from the histogram in Figure 15. Comparison of the fines contents from either the CPT or SPT split spoon samples of Units 2 and 3 demonstrates that the Unit 3 sands have less fines than those of Unit 2.

118. Since Unit 3 has thicker sand layers, it appears that this would be a good place for a site specific validation of the technique by which CPT data are used to estimate the SPT blowcounts. Discussions presented earlier stated that the distribution of SPT measured equivalent clean sand blowcounts, N_{1c}, in Unit 3 had a mean value of 24.3 blows/ft (See Figure 81). The CPT predicted mean value N_{1c} of 25.7 blows/ft compares favorably with that actually obtained from the SPT. This agreement demonstrates that the technique for estimating SPT blowcounts is very reasonable for the Unit 3 sands and improves the overall confidence in applying the technique to the Unit 2 sands.

119. A summary of the results of the statistical analyses for the CPT predicted blowcounts for Units 2 and 3 is listed in Table 10. Information from this analysis with respect to the equivalent clean sand blowcounts, N_{1c}, was used to estimate the cyclic and residual strengths of the sands in Units 2 and 3. The determination of cyclic strength of the foundation sands is discussed in the following section.

Estimation of Cyclic Strengths

120. The cyclic strengths of foundation Units 2 and 3 were estimated from the SPT and CPT predicted equivalent clean sand blowcounts, N_{1c} and the empirically derived charts developed by Seed. The SPT blowcounts were used for the foundation sands beneath the main embankment and the CPT predicted blowcounts were used in the switchyard area. The 30-percentile (mean minus one-half standard deviation) of the statistical distribution for the CPT predicted N_{1c} blowcounts was judged to be the appropriate level of conservatism

for the basis upon which to select cyclic strengths. Professor Seed recommended use of the 35-percentile of the CPT predicted blow counts (See Seed's letter of 3 February 1986 in Appendix A to Volume 1). The cyclic strengths obtained using Seed's chart in Figure 68 are applicable to conditions where the M - 7.5, the ground surface is level, and the vertical effective stress is equal to 1 tsf. The next section describes the factors that are used to extrapolate the chart strengths to conditions other than those listed above.

Modification factors to cyclic strength

121. The modification factors that need to be applied to the cyclic strengths determined from the chart for in situ conditions are shown below in Equation (4) and (5):

$$\left(\frac{\tau}{\sigma_v'}\right)_{\substack{\alpha \neq 0 \\ \sigma_v' \neq 0 \\ M \neq 7.5}} = K_M \times K_\sigma \times K_\alpha \times K_{layer} \times \left(\frac{\tau}{\sigma_v'}\right)_{\substack{\alpha = 0 \\ \sigma_v' = 1 \\ M = 7.5}} \qquad (4)$$

$$\tau_{str} = \sigma_v' \times \left(\frac{\tau}{\sigma_v'}\right)_{\substack{\alpha \neq 0 \\ \sigma_v' \neq 1\ tsf \\ M \neq 7.5}} \qquad (5)$$

where:

$\sigma_{v'}$ - vertical effective stress

α - initial shear stress ratio, the initial static horizontal shear stress (τ_{xy}) divided by the vertical effective stress ($\sigma_{v'}$)

K_M - adjustment factor to correct cyclic strength for different earthquake magnitudes and different numbers of cycles

$K\sigma$ - overburden correction factor to adjust cyclic strength for confining stress other than 1 tsf

$K\alpha$ - adjust factor to correct cyclic strength for non-zero initial horizontal static shear stress conditions

K_{layer} - adjustment factor to account for the inability of thin and layers to develop the full penetration resistance of the Cone Penetrometer.

$\left(\frac{\tau}{\sigma_{v'}}\right)_{\substack{\alpha = 0 \\ \sigma_{v'} = 1\ \text{tsf} \\ M = 7.5}}$ = cyclic shear stress ratio (liquefaction resistance) determined from empirical charts which correspond to $\sigma_{v'} = 1$ tsf, $\alpha = 0$, and $M = 7.5$.

$\left(\frac{\tau}{\sigma_{v'}}\right)_{\substack{\alpha \neq 0 \\ \sigma_{v'} \neq 1\ \text{tsf} \\ M \neq 7.5}}$ = cyclic shear stress ratio (liquefaction resistance) determined from empirical charts which correspond to $\sigma_{v'} \neq 1$ tsf, $\alpha \neq 0$, and $M \neq 7.5$.

The modification factors K_M, K_σ, and K_α are described in more detail in the following sections.

Modification factor for earthquake magnitude

122. The factor K_M accounts for the number of dynamic shear stress cycles the soil will experience during an earthquake of a particular magnitude. Larger numbers of cycles are associated with larger magnitudes and smaller numbers of cycles are associated with smaller magnitudes. The empirical charts have been developed for an earthquake of Magnitude 7.5. Table 11 shows the values of K_M associated with earthquake magnitudes that range from 5.25 to 8.5. The number of representative dynamic shear stress cycles are also shown in this table; they range from 2 to 3 for a magnitude 5.25 event to 26 cycles for a magnitude 8.5 event. For the Barkley site the design earthquake has a body-wave magnitude of M_b of 7.5. For the Seed cyclic strength chart, this equates to a Richter magnitude of 8.5 (Nuttli and Herrmann, 1982). Consequently, a K_M equal to 0.89 was used in the liquefaction analysis.

Modification factor for overburden stress

123. The overburden correction factor, K_σ, is applied to the cyclic strength to adjust for the nonlinear relationship between resistance to liquefaction and confining stress. The empirical charts give cyclic strengths that correspond to an effective confining stress of 1 tsf. The factor K_σ is used to adjust the chart values to confining stresses less than or greater than 1 tsf. The relationship between vertical effective stress and K_σ used in this study is shown in Figure 93.

Modification factor for
initial static shear stress

124. The factor K_α is applied to the cyclic shear strength to adjust for the increase in liquefaction resistance due to the presence of pre-earthquake static shear stresses, typically caused by nonlevel ground surfaces. The value α is the ratio the initial horizontal shear stress to the vertical effective stress. Both of these stresses were determined at various locations within the critical cross section from the static analysis using FEADAM in Part III of this report.

125. Based on the work of Szerdy (1986) and others (See Rollins, 1987, for summary) it is known that K_α is a function of relative density of the soil. For relative densities greater than 45 percent α values greater than zero will result in an increase in cyclic strength. For relative densities of approximately 45 percent α has no effect on the cyclic strength. For relative densities less than 45 percent the presence of initial horizontal shear stresses is detrimental and there is a reduction in cyclic strength. These trends are shown in Figure 94 and were taken from Rollins (1987). The equivalent sand N_{1c} in the Unit 2 sands average 19 blows per foot which correlates to a relative density of about 65 percent according to the chart in Figure 95 developed by *Tokimatsu and Seed*, 1987. In Unit 3 the equivalent sand N_{1c} is 25.7 blows/ft which correlates to a relative density about 75 percent. The K_α curve for a relative density of 55 percent in Figure 94 was used in this analysis to determine K_α for the sands in both Units 2 and 3.

Reduction factor for thin sand layers

126. As discussed earlier, the thin sand layers in Unit 2 posed a major problem with respect to measuring the "true" CPT tip resistance in these sands since the measurements were influenced to a large degree by the softer underlying clay soils. Professor H. Bolton Seed, a Technical Advisor to this project, recommended increasing the measured CPT tip resistances to account for the fact that the full penetration resistance may not be measured when thin sand layers are underlain by soft clays and to account for the uncertainty involved regarding the continuity of the individual sand layers in the soil profile (See Appendix D, Seed's letter dated 3 February 1986). As a result of Seed's recommendation the cyclic strengths determined from the CPT data of Unit 2 and from Seed's charts were increased by 25 percent (K_{1ayer} = 1.25) to account for these effects. This adjustment also reflects the belief that on

the average the compensating errors related to the CPT estimates of the soil classification and fines contents discussed in earlier in this Part did not exactly balance. Additionally, in Unit 3 where the sand layers were thicker, the cyclic strengths were not adjusted for the thinness and continuity of the sand layers and K_{layer} was equal to one.

Selection of cyclic strength
stress ratio for the sands in Unit 2

127. Switchyard area: The cyclic strengths in the switchyard area were determined from the CPT predicted blowcount data. The statistical analysis presented for the CPT predicted blowcounts presented earlier in this part of the report showed that the statistical distribution for N_{1c} had a mean value of 19.0 blows/ft and a standard deviation of 7.2 blows/ft (See Table 10). As stated earlier, the 30-percentile level of the N_{1c} distribution was adopted for selecting the cyclic strength stress ratio from Seed's chart in Figure 68. The 30-percentile is equal to the blowcount value which is one-half standard deviation less than the mean value. Thus the 30-percentile value for N_{1c} is:

$$\begin{aligned} N_{1c} &= 19.0 - (7.2/2) \\ &= 15.4 \text{ blows/ft, Say 15 blows/ft} \end{aligned}$$

Entering Figure 68 at 15 blows/ft results in a cyclic stress ratio of 0.165 for the Unit 2 sands. The cyclic strength stress ratio was corrected to a value of 0.184 after multiplying by the factors K_M (0.89) and K_{layer} (1.25) to account for the effects of a Magnitude 8+ event and the thin layers.

128. Main embankment area: The cyclic strengths of the Unit 2 sands beneath the main embankment area were determined directly from the statistical analysis of the SPT blowcounts presented earlier in Table 9. The statistical analysis indicated that the distribution of N_{1c} blowcounts had a mean value of 20.7 blows/ft and a standard deviation of 6.5 blows/ft. Thus, the 30 percentile value for N_{1c} was estimated to be:

$$\begin{aligned} N_{1c} &= 20.7 - 6.5/2 \\ &= 17.45 \text{ blows/ft, say 17.5 blows/ft} \end{aligned}$$

The cyclic strength stress ratio for a vertical effective stress of 1 TSF and level ground conditions was estimated to be 0.195 based on Seed's chart on

Figure 68. The cyclic strength stress ratio was corrected to a value of 0.174 after multiplying by the K_M factor of 0.89 to account for a Magnitude 8+ event.

Switchyard and main embankment areas: selection of cyclic stress strength ratio for the sands in Unit 3a and 3c

129. The cyclic strength of the Unit 3 sands in both the switchyard and main embankment areas were estimated from the CPT predicted N_{1c} blowcounts. In a similar manner as that for Unit 2, the statistical analysis of the distribution for the CPT predicted N_{1c} blowcounts of Unit 3 sands showed that the mean value was 25.7 blows/ft and that the standard deviation was 7.6 blows/ft. Thus, at the 30-percentile the equivalent sand blowcount, N_{1c}, was 21.9 blows/ft which was rounded to 22 blows/ft. From Seed's charts in Figure 68, 22 blows/ft results in a cyclic strength stress ratio of 0.241. The cyclic strength stress ratio was corrected to a value of 0.214 after multiplying by the K_M factor of 0.89 to account for a Magnitude 8+ event. In the switchyard area these results were applied to Units 3a and 3c which were the sandier portions of Unit 3. Unit 3b was treated as a nonliquefiable clay. In the main embankment area this cyclic strength was applied over the entire thickness of Unit 3.

Evaluation of Liquefaction Potential and Pore Pressure Generation Characteristics of the Sands in Foundation Units 2 and 3

General

130. Safety factors against liquefaction, FS_L, were computed to evaluate the liquefaction potential and pore pressure generation characteristics of the sands in Units 2 and 3 for the representative cross section of both the switchyard (See Figure 27) and main embankment areas (See Figure 54). The FS_L is defined as the ratio between cyclic strength and earthquake induced shear stress as expressed in Equation 6 below:

$$FS_L = \frac{\tau_{str}}{\tau_{dyn}} \tag{6}$$

131. For the switchyard area the FS_L's were computed at the centers of gravity for each of the foundation elements of the finite element mesh shown on Figure 34 for Units 2, 3a, and 3c. The cyclic strengths, τ_{str}, for each of these elements were evaluated using Equations 4 and 5. The cyclic stress ratios (adjusted for magnitude) estimated from the CPT predicted equivalent sand blowcounts were 0.184 for Unit 2 and 0.214 for Units 3a and 3c. Values of K_σ and K_α were determined from the vertical effective stress and alpha ratio (from the FEADAM analysis) and the relationships shown in Figures 93 and 94, respectively. Since the static and dynamic finite element meshes were different the static stresses were interpolated to positions of the centers of gravity on the dynamic mesh. The earthquake induced stresses, τ_{dyn}, determined from the dynamic analysis (See Part III) using FLUSH are shown in Figure 49. Having determined τ_{str} and τ_{dyn} the safety factors for each element were evaluated with Equation 6.

132. For the main embankment area, the FS_L's were computed at the centers of the layers for each of the four one-dimensional soil profiles shown in Figures 55 through 58. As for the switchyard area, the cyclic strengths were computed using Equations 4 and 5. The magnitude adjusted cyclic strengths used for the sands of Units 2 and 3 were 0.174 and 0.214, respectively. The K_{layer} factor for Unit 2 was set equal to one because the sands layers in Unit 2 beneath the main embankment are thicker than those beneath the switchyard. A K_{layer} factor equal to one was also assigned to Unit 3. The values of K_α were estimated based on the static finite element analysis of the switchyard area described in Part III. Static stress determined from the FEADAM analysis at locations upstream of the centerline were applied to corresponding locations on the upstream side of the main embankment because the upstream slope geometry and foundation conditions of the switchyard area approximate those of the main embankment. Values of K_σ and K_α were determined from the vertical effective stresses and α ratios (See Figures 30 and 32) and the relationships shown in Figures 93 and 94, respectively. The earthquake induced stresses used in the evaluation of the liquefaction potential for the foundation beneath the main embankment were presented in Figures 63 through 66.

133. In this analysis, the pore pressure generation characteristics of the sands in the Units 2 and 3 were related to values of safety factor against

liquefaction. The pore pressure generation characteristics are discussed in Part V of this report.

Computed factors of safety against liquefaction

134. Switchyard area: Contours of the computed safety factors against liquefaction were drawn on the idealized cross section and shown in Figure 96. The cross hatched zone on the drawing represent zones where FS_L values were less than one and enable visualization of the extent of the predicted zones of liquefaction. The method of analysis predicts that any sands located within the cross hatched areas will liquefy due to the motions of the design earthquake. In Figure 96, in Unit 2 liquefaction of the sands is predicted to occur in three basic areas: (a) from the downstream freefield to a location approximately 450 ft downstream of the centerline; (b) directly beneath the level section of the switchyard berm from about 325 ft to 125 ft downstream of the centerline; and (c) and from a location 100 ft upstream of the centerline and extending out to the upstream free field area. There will be two zones in Unit 2 where liquefaction is not predicted: (a) directly beneath the sloping section of the switchyard berm (between locations 450 and 325 ft downstream of the centerline) and (b) directly beneath the main portion of the embankment from locations about 125 ft downstream to 100 ft upstream of the centerline. Safety factors in the first reach have values which are as high as 1.2 and in the second are as high as 1.4.

135. In Unit 3, liquefaction is predicted in the sands of 3a and 3c at locations near the upstream and downstream freefields. The downstream zone of liquefaction extends from the free field area to about 500 ft downstream of the centerline. The upstream zone of liquefaction extends from the free field to about 225 ft upstream of the centerline. A significant portion of Unit 3 located between the two liquefiable zones is not predicted to liquefy. The FS_L values within this zone tend to increase in directions away from the two liquefiable areas and reach a maximum near the centerline where the FS_L contours reach values of about 1.4. As stated previously, the clay layer which comprises Unit 3b was treated as a nonliquefiable clay and safety factors against liquefaction were not computed for it.

136. Main embankment area: The results of the computed FS_L's of Units 2 and 3 are presented in Tables 12 through 16 for the one-dimensional profiles shown in Figure 54. All of the parameters and factors used in

evaluating the foundation FS_L's of each profile are listed in these tables. The values of alpha (determined from the static analysis) used for Profiles 1 though 3 are plotted versus depth in Figures 97 through 99, respectively. Values of alpha for Profile 4 at the centerline were equal to zero.

137. In this analysis, the dynamic stresses for Profile 1 were used to evaluate the FS_L's at locations in the free field and at the toe of the embankment as shown in Figure 56. At the location of the free field profile, the initial geostatic stresses are not influenced by the presence of the embankment. In this case, the FS_L's for each layer were computed by setting K_α equal to one to reflect level ground conditions (See Table 12). Under level ground conditions, shear stresses on horizontal planes are zero, hence, α is equal to zero. In the second case, the initial stresses are effected by the presence of the embankment and K_α was selected according to the stresses determined from the static analysis (See Table 13).

138. The safety factors against liquefaction from Tables 12 through 16 were synthesized into the cross section shown in Figure 100. The plot shows the average values for FS_L computed from layers in Units 2 and 3 for each profile. In Unit 2, the analysis indicates that liquefaction can be expected in both the upstream and downstream free field areas of the main embankment. The FS_L's improve to values ranging between 1.1 and 1.25 beneath the sloping sections of the embankment where the contribution of initial shear stresses have their greatest effect in increasing the cyclic strength of the foundation sands. Figure 100 shows that liquefaction is predicted in Unit 2 near the centerline where the FS_L's are only 0.9. Some liquefaction is predicted to occur in the upper reaches of the Unit 3 sands between elevations 285 and 295 ft beyond the upstream and downstream toes. Liquefaction is not expected in Unit 3 directly beneath the embankments where the analysis shows that FS_L's are greater than 1.3. A cross section of the main embankment showing the zones in the foundation where liquefaction was predicted are shown in Figure 101.

PART V: DETERMINATION OF POST-EARTHQUAKE STRENGTHS

General

139. In this part, the strengths postulated to exist in the various zones of the foundation immediately after the end of the earthquake were determined for input to the post-earthquake stability analysis. The post-earthquake strengths were selected by independent consideration of the characteristics of Units 2 and 3 (including subunits). Post-earthquake strengths were first selected for each of the major material types in the foundation which include: the normally consolidated clays of Units 2 and 3b and the sands of the Units 2 and Units 3a and 3c. Finally, a set of criteria was adopted which lead to conservative choices in the assignment of the post-earthquake strengths of the foundation soils in the in the idealized cross section. The criteria adopted for this study for each of the foundation units are discussed in the sections which follow. Ultimately, post-earthquake strengths were recommended for the foundation soils of two cross sections: the idealized cross sections of the switchyard (Figure 28) and the main embankment (See Figure 54). A stability analysis was performed on the representative sections of each area in Volume 5 using the recommended post-earthquake strengths.

Post-Earthquake Strengths

Normally consolidated clays

140. The clayey portions of Units 2 and 3b were determined to be normally consolidated based on the results of the Cone Penetration Testing (See Part II). Normally consolidated clays typically have c/p ratios equal to about 0.31. If the clays are assumed to lose 20 percent of their strength due to the earthquake shaking the post-earthquake c/p ratio will be equal to 0.25 (0.31 x 0.80) (Seed and Thiers 1968). Thus, according to criteria to be described later elements determined to be controlled by the criteria for clay were assigned the post-earthquake c/p ratio of 0.25.

Sands having
FS_L less than one

141. Sands which have a safety factor, FS_L, less than one are assumed to have liquefied due to the motion of the design earthquake. The post-earthquake strengths of the liquefied sands in Units 2 and Units 3a and 3c were estimated from the CPT predicted equivalent sand blowcounts, N_{1c}. A performance based chart that relates SPT blowcounts to post-earthquake strength for (materials that liquefied) evaluation of post-earthquake slope stability was introduced by Seed (1986). This chart, shown in Figure 102, was developed from back-calculated undrained strengths, S_{ur}, required to produce a safety factor of one against sliding in embankments that have failed by sliding due to liquefaction. These strengths were related to SPT blowcounts made at the sites. The data from which the chart was developed came primarily from sites having sands and silty sands.

142. Seed's work (1986) demonstrated that the correlation between SPT blowcounts and S_{ur} were dependent upon the fines content of the liquefied soil. Thus, the effective blowcount value, N_{1eff}, upon which the S_{ur} value of a liquefied soil is based is arrived at by adjusting the overburden corrected blowcount, N_1, for the percentage of material passing the No. 200 sieve. The fines correction made for the determination of S_{ur} is different than that made for cyclic strength discussed in Part IV. The value for N_{1eff} is determined using the following equation:

$$N_{1eff} = (N_1)_{Meas} + \Delta N_1 \tag{7}$$

where:

N_{1eff} = fines corrected blowcount used to determine the undrained residual strength of a liquefied soil.

$(N_1)_{meas}$ = measured blowcount corrected to a vertical effective stress of 1 tsf. In this study, $(N_1)_{meas}$ was assumed to be equal to $(N_1)_{60}$.

ΔN_1 = correction for fines content.

Values for ΔN_1 for different fines contents are listed in Table 17. Conservatively, the lower bound curve in Figure 102 was used to estimate the S_{ur} values from the N_{1eff} blowcounts for the liquefied sands in Units 2 and 3 in both the switchyard and main embankment areas.

143. Switchyard area: The residual strengths of the liquefied sands in Units 2 and 3 of the switchyard area were based upon the mean CPT predicted values for $(N_1)_{60}$ and fines contents listed in Table 18. The use of the mean value was justified because no correction was applied to increase the residual strength to account for the effect of thin layers on the penetration resistance as was done for the cyclic strength discussed previously. The CPT predicted fines contents rather than the SPT predicted fines contents were used in the main embankment area because their use resulted in a smaller ΔN_1 which yielded a conservative value for S_{ur}. Thus, the S_{ur} of liquefied sands in Unit 2 was determined from the chart on Figure 102 to be 450 psf based on an N_{1eff} blowcount of 15.5 blows/ft. In Unit 3 of the switchyard, the S_{ur} value was determined to be 800 psf based on an N_{1eff} value of 25.0 blows/ft. This choice is conservative in that it does not extrapolate beyond Seed's basic set of data.

144. Main embankment area: The post-earthquake undrained residual strength, S_{ur}, of the liquefied sands of Unit 2 was determined using mean values of the SPT blowcounts and fines contents determined in the statistical analysis discussed in Part IV (See Table 8). The data upon which S_{ur} was based are presented in Table 19. Based on an $(N_1)_{60}$ blowcount of 15.8 blows/ft and a fines content of 23.1 percent N_{1eff} was estimated to be 17.5 blows/ft. Thus, from Figure 102 the S_{ur} of the liquefied sands of Unit 2 was estimated to be 700 psf. The S_{ur} of Unit 3 in the main embankment area was estimated to be 800 psf on the same basis as that for the switchyard area.

Sands having FSL greater than one: pore pressure generation characteristics

145. Pore pressure generation can occur in sands even though the deposit does not liquefy. Tokimatsu and Yoshimi (1983) showed that the earthquake induced residual excess pore pressures (normalized with respect to vertical effective stress) are related to the stress ratio (ratio of actual shear stress ratio to the stress ratio causing liquefaction) as shown on Figure 103. Tokimatsu and Yoshimi found that for most sands the relationship between normalized parameters falls within the shaded area in the figure. The FS_L is represented by the reciprocal of the normalized stress ratio on the abscissa. In this analysis, the elements with FS_L greater than one were assigned excess pore pressure ratios, r_u, according to the relationship defined by the dashed

line in the center of the shaded area on Figure 103. In this study r_u , is the ratio of excess pore pressure (u_e) vertical effective stress as defined by Equation 8 below:

$$r_u = \frac{u_e}{\sigma'_v} \tag{8}$$

146. The excess pore-pressures generated by the earthquake was essential information to be input to the post-earthquake seismic stability analysis documented in Volume 5 of this series. In Units 2, 3a, and 3c, the strengths of nonliquefied sands were determined from the r_u value, the vertical effective stress, and a friction angle of 31 deg. A friction angle of 31 deg was used for Unit 2 to evaluate the stability in the initial design of the dam (US Army Engineer District-Nashville 1960).

Switchyard Area Cross-section

147. Post-earthquake strengths were assigned to each element according to criteria developed specifically for each of foundation Units 2, 3a, 3b, and 3c. These criteria account for the material types found in each unit and try to be conservative as to the final strength selected for each element. The criteria used for each of the foundation units are outlined in the following paragraphs.

Criteria for Unit 2

148. The criteria for foundation Unit 2 recognized that within any element there were both sands and clays as determined in Part II of this report. Hence, depending on the conditions it was possible to assign either the residual strength, S_{ur}, or a Mohr-Coulomb strength (friction ratio of 31 deg with a r_u value) for sands, or a strength based on the post-earthquake c/p ratio of 0.25 for normally consolidated clays. The assignment of the proper earthquake strength first depended upon whether or not the sands in the element were determined to liquefy (See Figure 96). If the safety factor, FS_L, of the element was less than one (i.e. sands liquefied), the strength assigned to the element was the lower of the residual strength (S_{ur}= 450 psf) and the undrained strength, c , based on a post-earthquake c/p ratio of 0.25. The

c/p based strength was determined by multiplying the vertical effective stress by 0.25. If the safety factor, FS_L , was greater than one and if the S_{ur} was greater than the Mohr-Coulomb based strength then the post-earthquake strength was based on the minimum value of the c/p based strength and the S_{ur} . This criteria controlled for cases where the safety factor was only marginally greater than one and guaranteed the element would be assigned a post-earthquake strength at least as great as that of either the S_{ur} or the c/p based value. If the safety factor, FS_L , was greater than one and if the S_{ur} was less than the Mohr-Coulomb strength than the post- earthquake strength was assigned based on the smaller of the Mohr-Coulomb and c/p based strengths. This criteria controlled for FS_L values which were more than nominally greater than one.

Criteria for Units 3a and 3b

149. These units were treated as consisting only of sands and thus the logic for assigning post-earthquake strength is somewhat simpler than that for Unit 2. Units 3a and 3b were treated identically. Again, the criteria depended on the whether or not the sands in Units 3a and 3c are predicted to liquefy. If the FS_L is less than one the post-earthquake strength is assigned the undrained residual strength value, S_{ur} , of 800 psf which was determined based on the CPT predicted blowcounts. If the value of FS_L was greater than one then the post-earthquake strength assigned to the element under consideration was the minimum of the Mohr-Coulomb based strength and the residual strength value. This criteria guaranteed that the strength in these units would be at least as large as the S_{ur} for safety factors only slightly greater than one. The Mohr-Coulomb strength of these sands was a friction angle of 31 deg and the r_u value for the estimated value of FS_L.

Criteria for Unit 3b

150. Unit 3b was determined to consist of normally consolidated nonliquefiable clays. Hence, the strengths were based on the post earthquake c/p ratio of 0.25 only.

Results

151. The post-earthquake strengths ultimately assigned to the switchyard cross section upon which the finite element analysis was based are shown in Figure 104. Zones (groups of elements) where the undrained residual strength controlled are indicated by the value of S_{ur} . Zones where the post-earthquake c/p ratio governed are also indicated by the c/p value

of 0.25. Areas in the foundation where the Mohr-Coulomb strengths controlled are indicated by the excess pore pressures values, r_u , to be assigned to these areas. Friction angles of 31 and 35 deg were assigned to these areas in Units 2 and 3, respectively. The strengths shown on Figure 104 are recommended for the post-earthquake stability analysis of the switchyard section.

Main Embankment Section

152. The one-dimensional dynamic response and liquefaction analysis of the four profiles of the main embankment were discussed previously in Part IV. The results were presented in terms of the safety factors against liquefaction, FS_L , given in Tables 12 through 15 and Figures 100 and 101. The analysis showed that liquefaction was predicted in the upstream and downstream free field and in the area immediate to the centerline of Unit 2. Liquefaction was not predicted in Unit 2 in areas directly beneath the slopes of the embankment where FS_L's ranged from about 1.1 to 1.25. Liquefaction was predicted to occur in the upper reaches of Unit 2 of the free field between elevations 285 and 295. The FS_L's in all other areas of Unit 3 were greater than 1.2 (free field) and 1.3 (beneath the embankment).

153. The liquefaction analysis indicates that widespread liquefaction of Units 2 and 3 is not expected beneath the main embankment. Nonetheless, as a conservative measure undrained residual strengths are recommended for both Units 2 and 3 in the post-earthquake stability analysis. The recommended strengths are presented on the cross-section on Figure 105. An undrained residual strength, S_{ur} , of 700 psf is recommended for the entire breadth of Unit 2 and 800 psf is recommended for Unit 3.

PART VI: SUMMARY AND CONCLUSIONS

154. This report describes the procedures used to interpret the data obtained from field investigations to determine the liquefaction potential and post-earthquake strengths of the foundation soils at Barkley Dam. The basic components of this report include the interpretation of the field data to evaluate the stratigraphy of the foundation and characterize the site, dynamic response analyses to evaluate the performance of the foundation of two representative embankment sections, the analysis and interpretation of Cone and Standard Penetration Test data to determine the cyclic strengths for evaluation of the potential for liquefaction and the post-earthquake strengths of the foundation soils. The post-earthquake strengths were recommended for input to the seismic stability analysis documented in Volume 5.

155. The stratigraphy analysis was based on data collected from Standard Penetration Tests (SPT), Cone Penetration Tests (CPT), undisturbed samples, and an excavation of an exposure of a critical foundation material. The data were analyzed in order to clarify the conditions in the foundation at the dam site. The strengths and weaknesses of all the basic sources of information were considered in arriving at an idealized interpretation of the site. The analysis showed that the foundation could be modeled by three basic foundation units: Unit 1, Unit 2, and Unit 3. The overall foundation thickness is approximately 120 ft. The data showed that each unit had traits which distinguished it from the others. Unit 1 consists primarily of clays which classify as CL materials. These clays which can be viewed as a topstratum are overconsolidated due to desiccation. Generally speaking, Unit 1 is thirty feet thick and lies between the elevations of 350 and 320 ft. Unit 2 consists of sand layers interbedded within a matrix of soft normally consolidated clay (CL). The sand layers are generally very thin being on the order of only a few inches thick. These sand layers are dirty and contain a fairly high percentage of nonplastic fines (on the order of 30 percent). Unit 2 generally lies between elevation 320 and 305 in the switchyard area and is a little thicker beneath the main dam where it extends to approximately elevation 295 ft. Unit 3 is subdivided into three subunits: Units 3a, 3b, and 3c. Unit 3a contains dense sands and gravels which have a relatively high penetration resistance as compared to the sands layers in Unit 2. These sands typically contain less than 15 percent fines and are relatively clean compared to

those in Unit 2. Unit 3a generally lies between elevation 305 and 295 in the switchyard area. Unit 3b, located between elevation 295 and 288, is typified by clays (CL) of low penetration resistance. Unit 3c lies between elevation 288 and bedrock and based on limited amounts of data appears to have characteristics similar to Unit 3a. Top of bedrock is generally encountered between elevation 230 and 240 beneath the switchyard and main embankment areas.

156. Preliminary studies showed that the sandy materials in foundation Unit 2 had a high potential for liquefaction in the event of the design earthquake and would be the main focus of the investigation. Thus, in the stratigraphy evaluation it was essential not only to identify and classify these materials but to attempt to their map lateral extent. Unfortunately, it was not possible to map the extent of individual sand layers from one CPT or SPT sounding to another. However, an excavation in a downstream exposure of Unit 2 revealed that some of these layers were undulating and continuous over fairly long distances in the direction of river flow. This result was carried into the liquefaction studies where the sandy materials in Unit 2 were conservatively treated as continuous.

157. The dynamic response of two representative cross sections were conducted to evaluate their overall performance due to the design earthquake. One cross section was located in the switchyard area and the second was representative of the main embankment area between Sta 43+00 and Sta 88+00. A two dimensional dynamic finite element analysis was performed using the computer program FLUSH to calculate the response of the representative switchyard section. This cross section was deemed critical due to the low SPT blowcounts that were measured in Unit 2 in the area of the switchyard. The cross section was excited with the motions generated by a magnitude (m_b) = 7.5 event based on the 1811-12 New Madrid event. The design accelerogram was scaled to a peak acceleration of 0.24 g and applied to the ground surface of a firm soil site. The S48°E component of the Santa Barbara Courthouse record of the Kern County Earthquake of 21 July 1952 was used as the input accelerogram. The dynamic earthquake induced stresses to be used in the evaluation of the liquefaction analysis were computed with FLUSH. The results of the analysis predict that large levels of cyclic strain which are on the order of about 1 percent are expected to occur in Unit 2 due to the design motions. This zone of large strains extends completely across the section from the downstream free field

to the upstream free field. A comparison of the dam's pre- (0.75 sec) and effective earthquake fundamental periods (1.75 sec) with the response spectra of the input accelerogram shows that the dam will resonate if subjected to these motions. The lengthening of the period indicates that significant strain softening of materials will occur in the foundation units. A series of one-dimensional dynamic response calculations using the computer code SHAKE were performed to approximate the earthquake induced stresses in the representative section of the main embankment. These analyses indicated that large strains could be expected to develop in Unit 2 due to earthquake shaking.

158. The cyclic strengths were determined from Seed's empirically based approach which uses SPT blowcounts. However, due to the complex nature of the thin interbedded sand lenses in Unit 2 - good estimates of the "true" SPT blowcounts were not attainable in the switchyard area where the critical section was located. Hence, a technique developed by Olsen (1984) was used to predict the SPT blowcounts from CPT test data. The CPT offered the advantage of high resolution and excellent sensitivity in the detection of the thin sand lenses.

159. For both the main embankment and switchyard section, the liquefaction potential was evaluated by computation of safety factors with respect to liquefaction in which the cyclic strengths are compared with the dynamic shear stresses. The results for the switchyard section are presented in Figure 96. Liquefaction is predicted in the foundation sands of Unit 2 over three basic areas: the upstream free field, beneath the switchyard berm and the downstream free field. Liquefaction is not expected to occur in Unit 2 just under the sloping section of the embankment and in the area near the centerline of the dam. Liquefaction is predicted in the sands of Unit 3 in the upstream and downstream free field areas, but is not expected to occur in Unit 3 sands directly beneath the embankment.

160. The results of the liquefaction analysis of the main embankment area are presented in Tables 12 through 16 and in Figures 100 and 101. These analyses showed that liquefaction of Unit 2 was predicted in the upstream and downstream free field and in the area under the centerline. Liquefaction was not predicted in Unit 2 in areas directly beneath the slopes of the embankment. Liquefaction was also expected in the upper reaches of Unit 2 of the free field between elevations 285 and 295. Liquefaction was not expected in all other areas of Unit 3 and in Unit 1.

161. Post-earthquake strengths were determined for both the switchyard and main embankment sections. The strengths were determined based on the results of the liquefaction analysis and the CPT predicted and SPT blowcounts. Seed's empirical technique which uses SPT blowcounts was used to estimate undrained residual strengths in areas where liquefaction of the sands was predicted. Earthquake indu ed excess pore pressures and strengths of clays were determined for areas where liquefaction is not expected. The post-earthquake strengths recommended for use in the stability analysis for the switchyard section are presented in Figure 104. The recommended post-earthquake strengths for the main embankment section are presented on Figure 105.

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Table 1

Undisturbed Borings

Boring No.	Date Drilled	Location: Longitudinal Station (ft)	Location: Sta from Centerline (ft)	EL. Top of Hole (ft)	Depth Soil (to rock) (ft)	No. of Samples
BEQ-1U	3 Nov 1977	64+20	2+31	349.6	114.2 (127.2)	39 Soil 2 Rock
BEQ-2U	Nov 1977	64+00	2+51	350.2	114.2 (121.9)	37 Soil 1 Rock
DS-1	23 May 1979	63+80	2+31	349.2	114.3 (122.0)	39 Soil 1 Rock
DS-2	2 Jun 1979	63+60	2+31	350.5	118.3 (124.4)	39 Soil 1 Rock
DS-3	9 Jun 1979	34+30	4+81	340.3	86.3 (94.1)	16 SPT 16 Soil
BEQ-3U	5 Dec 1984	37+00	0+44	385.2	47.6 --	5 Soil
BEQ-4U	31 Oct 1984	37+20	1+50	366.2	79.3 --	19 Soil
BEQ-5U	8 Nov 1984	34+61	4+86	341.7	54.7 --	15 Soil
BEQ-6U	14 Nov 1984	34+74	4+96	341.9	41.1 --	13 Soil
BEQ-7U	29 Nov 1984	37+00	5+20	347.7	78.7 --	21 Soil
BEQ-8U	16 Nov 1984	34+43	4+70	341.6	38.1 --	6 Soil

Table 2

SPT Lab Test Results

boring number	sample	top sample	bottom sample	nat. water (Wn)	liq. lim. (LL)	pls. lim. (PL)	D_60	D_50	D_30	D_10	% pass #200	% pass .005	word classification (minor)	word class. (major)	USCS soil class	color (minor)	color (major)
** BORING GROUP BEQ-10																	
BEQ-10	001A	0.00	0.50	23.70	25	16	0.031	0.019	.007	.002	67.9	24.5	GRAVELLY	CLAY	CL		
BEQ-10	001B	0.50	1.10	18.20	37	18	0.011	0.008	.002	-1.0	96.2	42.0		CLAY	CL		
BEQ-10	002	1.50	2.70	21.50	40	17	0.008	0.004	-1.0	-1.0	97.6	33.0		CLAY	CL		
BEQ-10	003A	3.00	3.30	27.20	0	0	0.005	0.002	-1.0	-1.0	98.9	38.0		CLAY	CL		
BEQ-10	003C	3.80	4.30	27.30	40	20	0.007	0.004	-1.0	-1.0	98.4	32.0		CLAY	CL		
BEQ-10	004A	4.50	5.00	21.30	38	20	0.010	0.007	.002	-1.0	96.8	43.0		CLAY	CL		
BEQ-10	004B	5.00	5.50	23.40	36	18	0.014	0.011	.003	-1.0	93.2	32.0		CLAY	CL		
BEQ-10	005	6.00	6.60	29.20	34	18	0.013	0.010	.003	-1.0	97.1	33.0		CLAY	CL		
BEQ-10	006B	8.00	8.30	23.40	0	0	0.013	0.009	.003	-1.0	95.0	36.0		CLAY	CL		
BEQ-10	007A	9.00	9.50	25.90	35	17	0.011	0.008	.002	-1.0	95.1	36.5		CLAY	CL		
BEQ-10	007B	9.50	10.00	24.20	30	17	0.020	0.015	.008	.001	95.4	23.0		CLAY	CL		
BEQ-10	007C	10.00	10.20	22.30	0	0	0.018	0.013	.007	.002	93.8	24.5		CLAY	CL		
BEQ-10	008A	10.60	11.10	22.40	30	17	0.014	0.010	.004	-1.0	96.9	32.5		CLAY	CL		
BEQ-10	008B	11.10	11.70	0.00	0	0	0.105	0.083	.040	.003	44.6	12.9	CLAYEY	SAND	SC		
BEQ-10	009A	12.00	12.60	22.90	27	19	0.040	0.024	.011	.001	83.1	21.0	SANDY	CLAY	CL		
BEQ-10	012B	12.60	13.30	24.30	34	17	0.032	0.022	.009	.001	80.7	21.0	SANDY	CLAY	CL		
BEQ-10	013A	13.50	13.70	25.50	25	17	0.037	0.024	.011	-1.0	78.0	21.0	SANDY	CLAY	CL		
BEQ-10	013B	13.70	14.30	22.50	0	0	0.070	-1.00	-1.0	-1.0	62.3	-1.0	SANDY	CLAY	CL		
BEQ-10	013C	14.30	14.80	25.70	0	0	0.070	-1.00	-1.0	-1.0	65.0	-1.0	SANDY	SILT	ML		
BEQ-10	014A	15.00	15.50	0.00	0	0	0.031	0.021	.008	-1.0	82.7	23.0	SANDY	CLAY	CL		
BEQ-10	015A	16.50	17.00	0.00	0	0	0.140	0.105	.024	.002	41.5	13.0	CLAYEY	SAND	SC		
BEQ-10	015B	17.00	17.50	23.80	0	0	0.135	0.091	-1.0	-1.0	45.0	-1.0	SILTY	SAND	SM		
BEQ-10	016A	18.00	18.60	24.90	0	0	0.068	0.030	.011	.001	61.2	20.5	SANDY	CLAY	CL		
BEQ-10	016B	18.80	19.10	24.80	0	0	0.115	0.100	.074	-1.0	31.1	-1.0	SILTY	SAND	SM		
BEQ-10	017A	19.50	20.00	24.10	23	16	0.074	0.035	.013	.002	60.2	18.5	SANDY	CLAY	CL		
BEQ-10	018A	21.00	21.70	0.00	31	19	0.022	0.017	.008	.001	88.2	23.0		CLAY	CL		
BEQ-10	018B	21.70	22.50	25.60	31	18	0.023	0.016	.010	.007	84.0	22.0	SANDY	CLAY	CL		
BEQ-10	019A	22.50	23.20	0.00	33	18	0.023	0.019	.007	.001	89.6	25.0		CLAY	CL		
BEQ-10	019B	23.20	24.00	26.60	0	0	0.019	0.013	.005	.001	87.4	30.5		CLAY	CL		
BEQ-10	020A	24.00	24.60	0.00	33	18	0.030	0.021	.010	.002	80.9	18.0	SANDY	CLAY	CL		
BEQ-10	020B	24.60	25.20	25.50	34	18	0.023	0.016	.007	-1.0	87.3	25.0	SANDY	CLAY	CL		
BEQ-10	022A	25.50	26.10	25.20	24	17	0.065	0.035	.012	.001	61.2	19.5	SANDY	CLAY	CL		
BEQ-10	022B	26.10	26.70	27.30	30	18	0.030	0.020	.007	-1.0	82.5	23.5	SANDY	CLAY	CL		
BEQ-10	022C	26.70	27.00	24.60	0	0	0.185	0.170	.131	-1.0	14.0	-1.0	SILTY	SAND	SM		
BEQ-10	023A	27.00	28.00	23.70	0	0	0.038	0.034	.014	.002	66.7	18.5	SANDY	CLAY	CL		
BEQ-10	023B	28.00	28.50	21.40	0	0	0.155	0.135	.080	-1.0	27.2	-1.0	SILTY	SAND	SM		
BEQ-10	024	28.50	29.70	0.00	0	0	0.170	0.150	.110	-1.0	21.0	-1.0	SILTY	SAND	SM		
BEQ-10	025	30.60	31.40	0.00	0	0	0.180	0.160	.130	-1.0	4.7	-1.0		SAND	SP		
BEQ-10	026A	31.50	32.00	30.80	0	0	0.051	0.036	.011	.001	73.5	19.5	SANDY	CLAY	CL		
BEQ-10	026B	32.70	33.00	35.30	35	21	0.051	0.030	.007	.002	70.1	21.0	SANDY	CLAY	CL		
BEQ-10	027B	33.90	34.50	0.00	0	0	0.170	0.150	.110	-1.0	17.9	-1.0	SILTY	SAND	SM		
BEQ-10	028A	34.50	35.20	33.30	0	0	0.079	0.052	.013	.003	59.0	15.0	SANDY	CLAY	CL		
BEQ-10	028B	35.20	36.00	21.70	0	0	0.120	0.092	.017	.002	45.8	16.0	SILTY	SAND	SM		
BEQ-10	029A	36.00	36.90	26.90	0	0	0.071	0.052	.010	.001	60.6	19.0	SANDY	CLAY	CL		

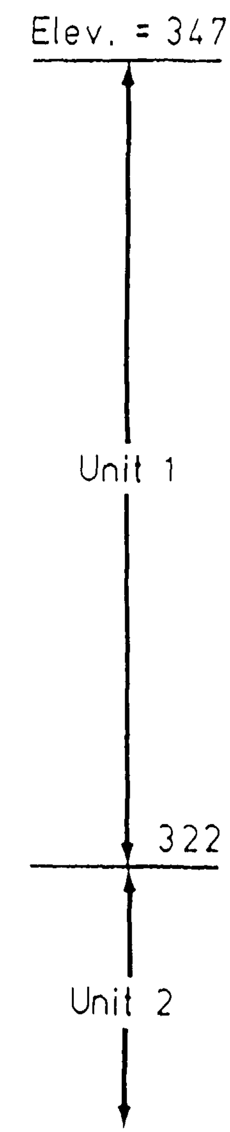

Table 2 (Concluded)

boring number	sam- pler	top sample	bottom sample	nat. water (Wn)	liq. lim. (LL)	pla. lim. (PL)	D_60	D_50	D_30	D_10	% pass #200	% pass .005	word classification (minor)	word class. (major)	USCS soil class	color (minor)	color (major)
BEQ-10	029B	36.90	37.50	28.90	0	0	0.041	0.021	.008	.001	76.6	23.0	SANDY	CLAY	CL		
BEQ-10	030A	37.50	38.20	28.80	32	18	0.054	0.041	.015	.002	72.9	15.5	SANDY	CLAY	CL		
BEQ-10	030B	38.20	39.00	27.60	33	18	0.014	0.009	.002	-1.0	83.7	39.0	SANDY	CLAY	CL		
BEQ-10	031A	39.00	39.70	29.60	34	19	0.028	0.019	.007	-1.0	78.0	24.0	SANDY	CLAY	CL		
BEQ-10	031B	39.70	40.50	28.30	32	17	0.059	0.040	.010	.001	66.5	24.5	SANDY	CLAY	CL		
BEQ-10	032A	40.50	41.20	26.50	0	16	0.053	0.032	.010	.001	67.5	24.0	SANDY	CLAY	CL		
BEQ-10	032B	41.20	42.00	30.00	31	16	0.048	0.025	.008	.001	68.1	26.5	SANDY	CLAY	CL		
BEQ-10	033	42.10	43.50	30.40	0	0	0.031	0.019	.005	-1.0	73.8	30.0	SANDY	CLAY	CL		
BEQ-10	034A	43.50	44.30	27.40	31	18	0.085	0.059	.019	.002	55.7	15.0	GRAVELLY SANDY	CLAY	CL		
BEQ-10	034B	44.30	44.60	0.00	0	0	3.300	0.900	.210	.130	2.0	0.5	GRAVELLY	SAND	SM		
BEQ-10	035A	45.00	45.80	26.90	32	18	4.000	0.110	.019	.002	45.1	15.5	SANDY CLAYEY	GRAVEL	GC		
BEQ-10	035B	45.80	46.50	28.70	35	18	0.025	0.016	.006	-1.0	75.8	27.5	SANDY	CLAY	CL		
BEQ-10	036A	46.50	47.30	26.80	30	17	0.030	0.021	.009	.001	75.1	23.0	SANDY	CLAY	CL		
BEQ-10	036B	47.30	48.00	27.60	30	16	0.025	0.015	.006	-1.0	71.5	28.0	SANDY	CLAY	CL		
BEQ-10	037A	48.00	48.80	30.40	38	20	0.020	0.014	.005	-1.0	76.9	28.5	SANDY	CLAY	CL		
BEQ-10	037B	48.80	49.50	25.30	33	18	0.021	0.015	.005	-1.0	76.1	28.5	SANDY	CLAY	CL		
BEQ-10	039A	51.00	51.80	31.50	39	20	0.014	0.090	.005	-1.0	96.0	29.5		CLAY	CL		
BEQ-10	039B	51.80	52.50	29.70	41	20	0.015	0.010	.004	.002	75.7	34.5	SANDY	CLAY	CL		
BEQ-10	040A	52.50	53.20	34.80	31	17	11.00	1.000	.105	.003	26.1	13.0	SANDY CLAYEY	GRAVEL	GC		
BEQ-10	040B	53.20	53.80	30.60	29	14	0.026	0.015	.005	-1.0	70.7	28.0	SANDY	CLAY	CL		
BEQ-10	041	54.50	55.50	19.90	0	0	0.330	0.300	.240	.130	8.8	-1.0		SAND	SP-SM		
BEQ-10	042	55.50	56.30	21.00	0	0	0.270	0.235	.210	.150	6.3	-1.0		SAND	SP-SM		
BEQ-10	043	57.00	57.60	26.10	0	0	0.235	0.225	.195	.145	4.8	4.5		SAND	SP		
BEQ-10	044	59.00	60.00	17.60	0	0	0.268	0.254	.210	.073	9.8	4.0		SAND	SP-SM		

Unit 2

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Unit 3

Table 3

Properties for Static Analysis Using FEADAM

		YOUNG'S MODULUS				BULK MODULUS						
MAT	Description	UNIT WT	K	KUR	N	RF	KB	M	C	ϕ	$\Delta\phi$	K_o
1	Random Fill	.0660	90.0	90.0	.45	.70	80.00	.20	.20	30.00	.00	.50
2	Compacted Embankment Fill	.1260	120.0	120.0	.45	.70	110.00	.20	.30	30.00	.00	.50
3	Unit 1 - Clay	.0660	120.0	120.0	.45	.70	110.00	.20	.30	30.00	.00	.50
4	Unit 2 and 3a - Silty Sand	.0640	300.0	300.0	.25	.70	250.00	.00	.00	32.00	4.00	.50
5	Unit 2 and 3b - Clay	.0640	300.0	300.0	.25	.70	250.00	.00	.00	32.00	4.00	.50
6	Unit 3c - Sands and Gravels	.0660	300.0	300.0	.40	.70	75.00	.20	.00	36.00	5.00	.40
7	Submerged Compacted Embankment Fill (Same as 2)	.0660	120.0	120.0	.45	.70	110.00	.20	.30	30.00	.00	.50

NOTES: 1. The Cohesion, C , is in TSF.
2. The unit weights fine in TSF.

Table 4

Material Properties for Dynamic Analysis

Material No.	Description	Total Density pcf	Shear Wave Velocity fps	Shear Modulus ksf
1	Random Fill	129	500	1001
2	Compacted Embankment Fill	126	600	1408
3	Unit 1 Lean Clay	129	775	2406
4	Units 2 and 3a Sand	127	700	1932
5	Unit 3b Clay	127	800	2525
6	Unit 3c Sand and Gravel	129	900	3245

Table 5

Criteria for Establishing Liquefaction Potential from SPT Data

If the Following Condition(s):	Then the Following:
Condition 1 Blow count < 10 Soil losses > 0.2 ft and Elevation > 300 ft	Soil lost is assumed to be classified as SM, 15 percent passing No. 200 sieve, D_{50} = 0.2, 0 percent passing 0.005
Condition 2 Count > 10 and Soil losses > 0.2 ft and Elevation > 300 ft	Soil lost is assumed to be blow classified as SP, 5 percent No. 200 sieve, D_{50} = 0.25 mm, 0 percent 0.005 mm size
Condition 3 Field classification is clay and no laboratory index tests	non-liquefiable by classification
Condition 4 Liquid limit > 35 percent	non-liquefiable by Chinese criteria
Condition 5 Percent passing 0.005 mm size > 15 percent	non-liquefiable by Chinese criteria
Condition 6 Classification is clay with $W_n < 0.9 \times LL$	non-liquefiable by Chinese criteria

Table 6

Example Showing the Results of SPT Data Analysis for Boring BEQ-30

BEQ-30
Barkley dam

Depth	Elev	EUS	Soil		(--%---			)	(------D50	------	)	(--P200	--	)	(-----	P005	----)	(SPT)		(-N1c--)
feet	feet	tsf	&	IC	PL	Un	LL	Un/LL	(--L	--Aver---	H--)	(L	-Aver-	H)	(--L	--Aver--	H-)	N	N1	P200
1.0	346.7	2.8	0	_														0	0	
25.0	322.7	1.5	3	C	14	21	27	0.79		0.04		53	60	72		24.8		7	6	
26.5	321.2	1.6	3	CS	14	21	25	0.85	0.06	0.08	0.11	44	51	56	22.3	24.0	24.8	6	5	
28.0	319.7	1.7	3	C		23						64	67	72				5	4	10.0
29.5	318.2	1.7	2		15	23	26	0.87	0.03	0.12	0.20	15	39	57		23.0		6	5	
31.0	316.7	1.8	3	CS		23			0.05	0.12	0.20	15	43	63		19.5		5	4	10.0
32.5	315.2	1.8	2	C		26				0.20		15	53	72				4	3	9.3
34.0	313.7	1.9	2			26				0.20		15	48	83				3	2	8.6
35.5	312.2	1.9	3	CS		24			0.11	0.14	0.20	15	56	77		17.5		6	4	10.5
37.0	310.7	2.0	3	C	15	24	24	1.01	0.02	0.07	0.20	15	49	63	21.0	22.6	24.5	3	2	
38.5	309.2	2.0	2	C	14	24	22	1.11	0.14	0.17	0.20	15	36	53		17.0		5	4	9.8
40.0	307.7	2.0	1	S		24			0.18	0.19	0.20	15	20	30		12.1		9	6	10.8
41.5	306.2	2.1	2	C	17	25	28	0.90	0.02	0.13	0.20	15	45	72		23.0		6	4	
44.5	303.2	2.2	3	C		25			0.01	0.11	0.25	5	51	83	30.0	33.9	37.0	13	9	
46.0	301.7	2.2	1	S		24			0.25	0.25	0.25	5	6	7				31	21	21.4
47.5	300.2	2.3	3	SC	32	41	58	0.71	0.24	0.25	0.25	5	38	90				10	7	
49.0	298.7	2.4	3	CS		25			0.32	5.41	10.50	3	26	56				49	32	32.5
50.5	297.2	2.4	1			21				6.90			5					37	24	24.2
52.0	295.7	2.4	2	S		17				0.27		12	14	16				48	31	32.5
53.5	294.2	2.5	2	S		25				0.19		14	15	15				21	13	17.7
55.0	292.7	2.5	3	SC		29						31	62	87				12	8	13.9
56.5	291.2	2.6	3	CS	17	30	34	0.88		0.02		69	72	77		23.0		6	4	
58.0	289.7	2.6	1	S		24				0.14		27	27	27				17	11	16.3
59.5	288.2	2.7	2	S		21			0.23	0.24	0.24	7	8	8				49	30	31.5
61.0	286.7	2.7	2	_		15						19	15	20				59	36	32.5
62.5	285.2	2.8	2	S		20				0.60		10	10	10				23	14	16.2

Table 7

Description of SPT Liquefaction Printout

Title	Description
Depth feet	Bottom 1/3 depth of 1.5 ft drive of SPT sampler (feet)
Elev feet	Elevation of SPT sampler depth
EVS (tsf)	Effective vertical stress (tsf)
No.	Number of soil samples in this SPT sampler
Soil ID*	Two letter soil classification based on soil sample code (major and minor) C clays M mixtures S sands (i.e. CM is mostly clay with some soil samples of mixture)
PL	Plastic Limit (averaged) in percent
Wn	Natural water content (W_n) (averaged) in percent
LL	Liquid Limit (averaged) in percent
Wn/LL	Water content divided by Liquid Limit
D50 Aver	Statistical D_{50} average of the soil samples
D50 L	The lowest D_{50} of the soil sample range
D50 H	The highest D_{50} of the soil sample range
P200 Aver	Statistical average of the percent passing the No. 200 sieve
P200 L	The lowest percent passing the No. 200 sieve
P200 H	The highest percent passing the No. 200 sieve
P005 Aver	Statistical average of the percent grain size of 0.005 mm
P005 L	The lowest of the percent grain size of 0.005 mm range
P005 H	The highest of the percent grain size of 0.005 mm range
SPT N	Measured SPT blowcount from 0.5 to 1.5 ft below the bore hole bottom
SPT N1	Calculated N_1 based on the C_n correction factor
N1c P200	N_{1c} silt correction based on the percent passing the No. 200 sieve

* Not to be confused with USCS classifications.

Table 8

Statistical Analysis of $(N_1)_{60}$ and Fines Content of SPT's Performed in Main Embankment Area

Location	Number of Samples	$(N_1)_{60}$ (Blows/ft)		Fine Content percent	
		Mean	STD	Mean	STD
Unit 2	82	15.8	8.3	23.2	16.2
Unit 3	48	23.8	8.9	16.1	10.6

Table 9

Summary of Statistical Analysis of Equivalent Sand Blowcounts, $(N_1)_c$, Obtained from Standard Penetration Tests

Area	Unit	Elevation	Mean $(N_1)_c$, blows/ft.	Standard Deviation	Number of Samples
Switchyard	2	305-320	13.6	3.0	22
Main Embankment	2	305-320	20.7	6.5	83
Total Site (Switchyard and Main Embankment)	3	Below 305	24.3	9.9	158

Table 10

Statistical Analysis of CPT Predicted Blowcounts

and Comparisons with SPT Test Results

	CPT Predicted Values					SPT Values*			
	$(N_1)_{60}$ blows/ft		N_{1c} blows/ft		Fines	N_{1c} blows/ft		Fines	
Unit	Mean	Standard Deviation	Mean	Standard Deviation	Percent	Mean	Standard Deviation	Percent	STD
2	14.5	8.9	19.0	7.2	15.0*	13.6	3.0	27.7	13.3
3	24.3	7.6	25.7	7.6	5.0*	24.3	9.9	15.9	12.0

* SPT values are for the total site which includes data from both the main embankment and the switchyard areas.

Table 11

Earthquake Magnitude Modification Factor, K_M, from Seed et al. (1983)

Earthquake Magnitude (M_L)	Number of Representative Stress Cycles (N)	Earthquake Magnitude Modification Factor K_m
8 1/2	26	0.89
7 1/2	15	1
6 3/4	10	1.13
6	5	1.32
5 1/4	2 to 3	1.5

Table 12

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1 at Upstream Free Field of Main Embankment

Layer No.	Depth	Elevation	Unit No.	σ_v' tsf	α	N_1	$\frac{\tau_{str}}{\sigma_v'}$ *	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS
6	27.5	322.5	2	0.92	0.23	17	0.187	0.89	1.04	1.00	318	384	0.82
7	32.5	317.5	2	1.08	0.22	17	0.187	0.89	0.94	1.00	338	447	0.76
8	37.5	312.5	2	1.25	0.20	17	0.187	0.89	0.92	1.00	382	501	0.76
9	42.5	307.5	2	1.40	0.19	17	0.187	0.89	0.90	1.00	419	553	0.75
10	47.4	302.5	2	1.56	0.17	17	0.187	0.89	0.87	1.00	451	598	0.75
11	55.0	294.5	3	1.80	0.15	22	0.242	0.89	0.84	1.00	651	632	1.03
12	65.0	284.5	3	2.12	0.14	22	0.242	0.89	0.80	1.00	731	641	1.14
13	75.0	274.5	3	2.45	0.12	22	0.242	0.89	0.74	1.00	781	693	1.13
14	87.5	262.0	3	2.85	0.11	22	0.242	0.89	0.72	1.00	883	691	1.28
15	102.5	247.0	3	3.35	0.11	22	0.242	0.89	0.71	1.00	1020	829	1.24

* Applies to conditions where $\alpha = 0$ and $\sigma_v' = 1$ tsf.

Table 13

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1 at Upstream Toe of Main Embankment

Layer No.	Depth	Elevation	Unit No.	σ_v' tsf	α	N_1	$\frac{\tau_{str}}{\sigma_v'}$ *	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS
6	27.5	322.5	2	0.92	0.23	17	0.187	0.89	1.04	1.76	560	384	1.95
7	32.5	317.5	2	1.08	0.22	17	0.187	0.89	0.94	1.73	585	447	1.31
8	37.5	312.5	2	1.25	0.20	17	0.187	0.89	0.92	1.69	647	501	1.29
9	42.5	307.5	2	1.40	0.19	17	0.187	0.89	0.90	1.67	700	553	1.26
10	47.4	302.5	2	1.56	0.17	17	0.187	0.89	0.87	1.63	736	598	1.23
11	55.0	294.5	3	1.80	0.15	22	0.242	0.89	0.84	1.54	1004	632	1.59
12	65.0	284.5	3	2.12	0.14	22	0.242	0.89	0.80	1.49	1089	641	1.70
13	75.0	274.5	3	2.45	0.12	22	0.242	0.89	0.74	1.44	1125	693	1.62
14	87.5	262.0	3	2.85	0.11	22	0.242	0.89	0.72	1.42	1255	691	1.82
15	102.5	247.0	3	3.35	0.11	22	0.242	0.89	0.71	1.42	1455	829	1.76

* Applies to conditions where $\alpha = 0$ and $\sigma_v' = 1$ tsf.

Table 14

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1 one-third of the Way Upslope of the Main Embankment

Layer No.	Depth	Elevation	Unit No.	σ_v' tsf	α	N_1	$\frac{\tau_{str}}{\sigma_v'}$ *	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS
9	40.5	322.5	2	1.42	0.22	17	0.187	0.89	1.70	1.70	707	619	1.14
10	45.5	317.5	2	1.58	0.20	17	0.187	0.89	1.68	1.68	768	686	1.12
11	50.5	312.5	2	1.75	0.20	17	0.187	0.89	1.68	1.68	819	737	1.11
12	55.5	307.5	2	1.91	0.20	17	0.187	0.89	1.68	1.68	876	785	1.12
13	60.5	302.5	2	2.07	0.18	17	0.187	0.89	1.65	1.65	910	820	1.11
14	68.0	295.0	3	2.31	0.16	22	0.242	0.89	1.60	1.60	1209	850	1.42
15	78.0	285.0	3	2.63	0.13	22	0.242	0.89	1.45	1.45	1215	856	1.42
16	88.0	275.0	3	2.96	0.13	22	0.242	0.89	1.45	1.45	1331	858	1.55
17	87.5	262.0	3	2.85	0.11	22	0.242	0.89	0.72	1.42	1255	691	1.82
15	100.5	262.5	3	3.36	0.13	22	0.242	0.89	1.45	1.45	1469	868	1.69
18	115.5	257.5	3	3.84	0.13	22	0.242	0.89	1.45	1.45	1833	868	1.89

* Applies to conditions where α = 0 and σ_v' = 1 tsf.

Table 15

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1 Two-thirds of the Way Upslope of the Main Embankment

Layer No.	Depth	Elevation	Unit No.	σ_v' tsf	α	N_1	$\frac{\tau_{str}}{\sigma_v'}$ *	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS
10	53.5	322.5	2	2.35	0.15	17	0.187	0.89	0.78	1.55	947	773	1.23
11	58.5	317.5	2	2.52	0.15	17	0.187	0.89	0.76	1.55	988	820	1.20
12	63.5	312.5	2	2.70	0.14	17	0.187	0.89	0.74	1.50	998	866	1.15
13	68.5	307.5	2	2.80	0.13	17	0.187	0.89	0.72	1.49	1000	903	1.11
14	73.5	302.5	2	3.00	0.12	17	0.187	0.89	0.71	1.45	1028	932	1.10
15	81.0	295.0	3	3.24	0.11	22	0.242	0.89	0.68	1.42	1347	957	1.41
16	91.0	285.0	3	3.60	0.10	22	0.242	0.89	0.66	1.38	1412	970	1.45
17	101.0	275.0	3	3.90	0.09	22	0.242	0.89	0.66	1.35	1496	986	1.52
18	133.5	262.5	3	4.30	0.09	22	0.242	0.89	0.64	1.35	1600	984	1.63
19	128.5	247.5	3	4.80	0.09	22	0.242	0.89	0.63	1.35	1759	1045	1.68

* Applies to conditions where $\alpha = 0$ and $\sigma_v' = 1$ tsf.

Table 16

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1 at the Centerline of the Main Embankment

Layer No.	Depth	Elevation	Unit No.	σ_v' tsf	α	N_1	$\frac{\tau_{str}}{\sigma_v'}$ *	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS
8	65.5	322.5	2	3.31	0	17	0.187	0.89	0.70	1.0	771	885	0.87
9	70.5	317.5	2	3.47	0	17	0.187	0.89	0.69	1.0	797	883	0.90
10	75.5	312.5	2	3.36	0	17	0.187	0.89	0.68	1.0	806	925	0.87
11	80.5	307.5	2	3.79	0	17	0.187	0.89	0.68	1.0	858	944	0.91
12	85.5	302.5	2	3.96	0	17	0.187	0.89	0.67	1.0	883	972	0.91
13	93.0	295.0	3	4.20	0	22	0.242	0.89	0.65	1.0	1175	1001	1.17
14	103.0	285.0	3	4.52	0	22	0.242	0.89	0.64	1.0	1246	1020	1.22
15	113.0	275.0	3	4.84	0	22	0.242	0.89	0.63	1.0	1313	1035	1.27
16	125.5	262.5	3	5.25	0	22	0.242	0.89	0.60	1.0	1357	1072	1.27
17	140.5	247.5	3	5.73	0	22	0.242	0.89	0.50	1.0	1457	1064	1.37

* Applies to conditions where $\alpha = 0$ and $\sigma_v' = 1$ tsf.

Table 17

Approximate Values of ΔN_1

Fines Content percent	ΔN_1 blows/ft
10	1
25	2
50	4
75	5

(Reference: Seed, 1986).

Table 18

Residual Undrained Strengths of Liquefied Sands in the Foundation of the Switchyard Area

Foundation Unit	CPT Predicted $(N_1)_{60}$ Blows/ft	CPT Predicted Fines Content percent	ΔN_1	$N_{1\ eff}$	S_{ur} psf
2	14.5	15	1	15.5	450
3a and 3c	24.3	7	0.7	25.0	800

Notes: S_{ur} values for Units 2 and 3 based upon CPT predicted SPT Data on Table 10.

Table 19

Residual Undrained Strengths of Liquefied Sands in the Foundation of the Main Embankment Area

Foundation Unit	$(N_1)_{60}$ Blows/ft	Fines Content percent	ΔN_1	$N_{1\ eff}$	S_{ur} psf
2	15.8	23.1	1.7	17.5	700
3	24.3	7	0.7	25.0	800

Notes: 1. S_{ur} values for Unit 2 based upon SPT Data on Table 8.
2. S_{ur} values for Unit 3 based upon CPT Data on Table 10.

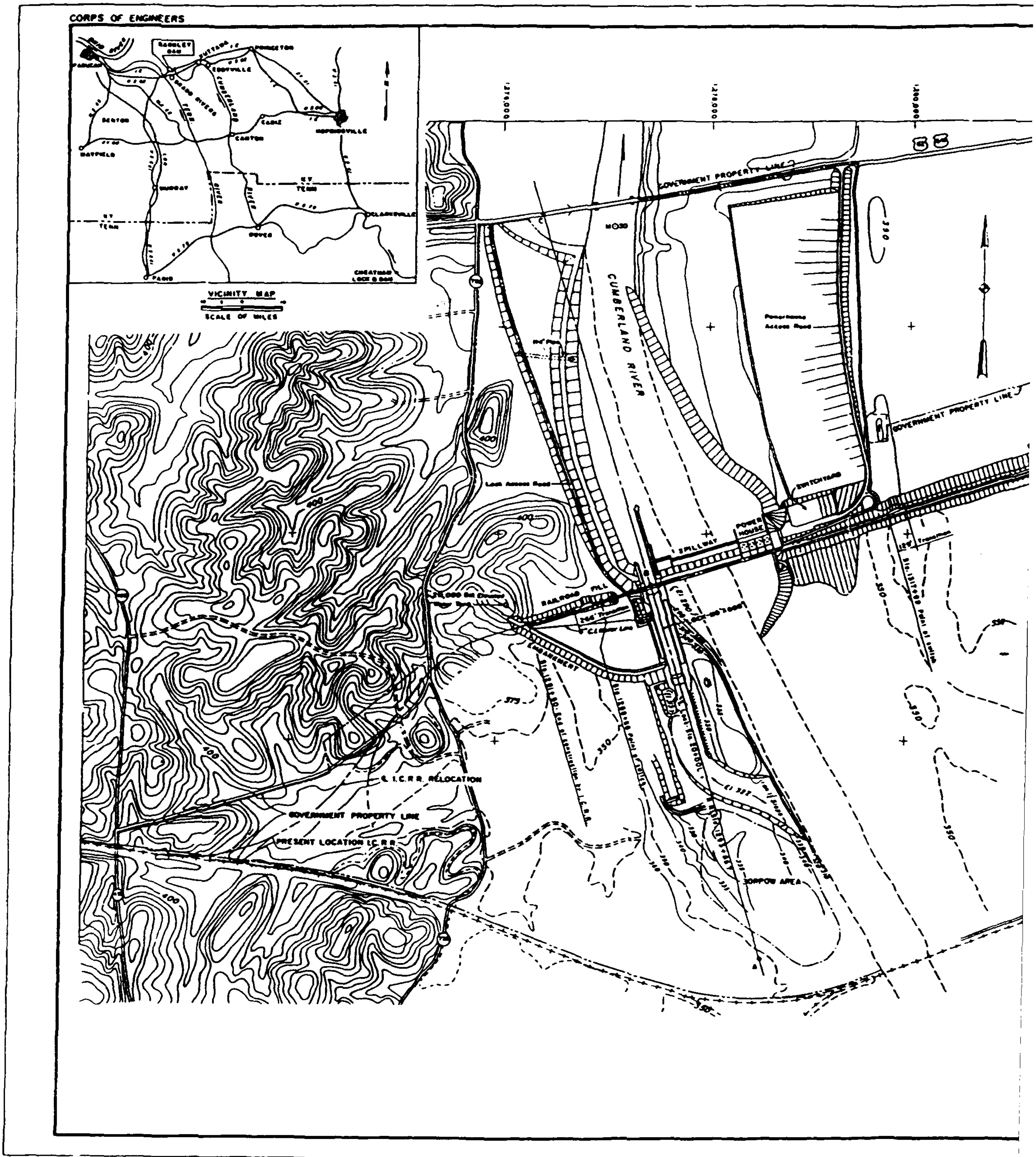
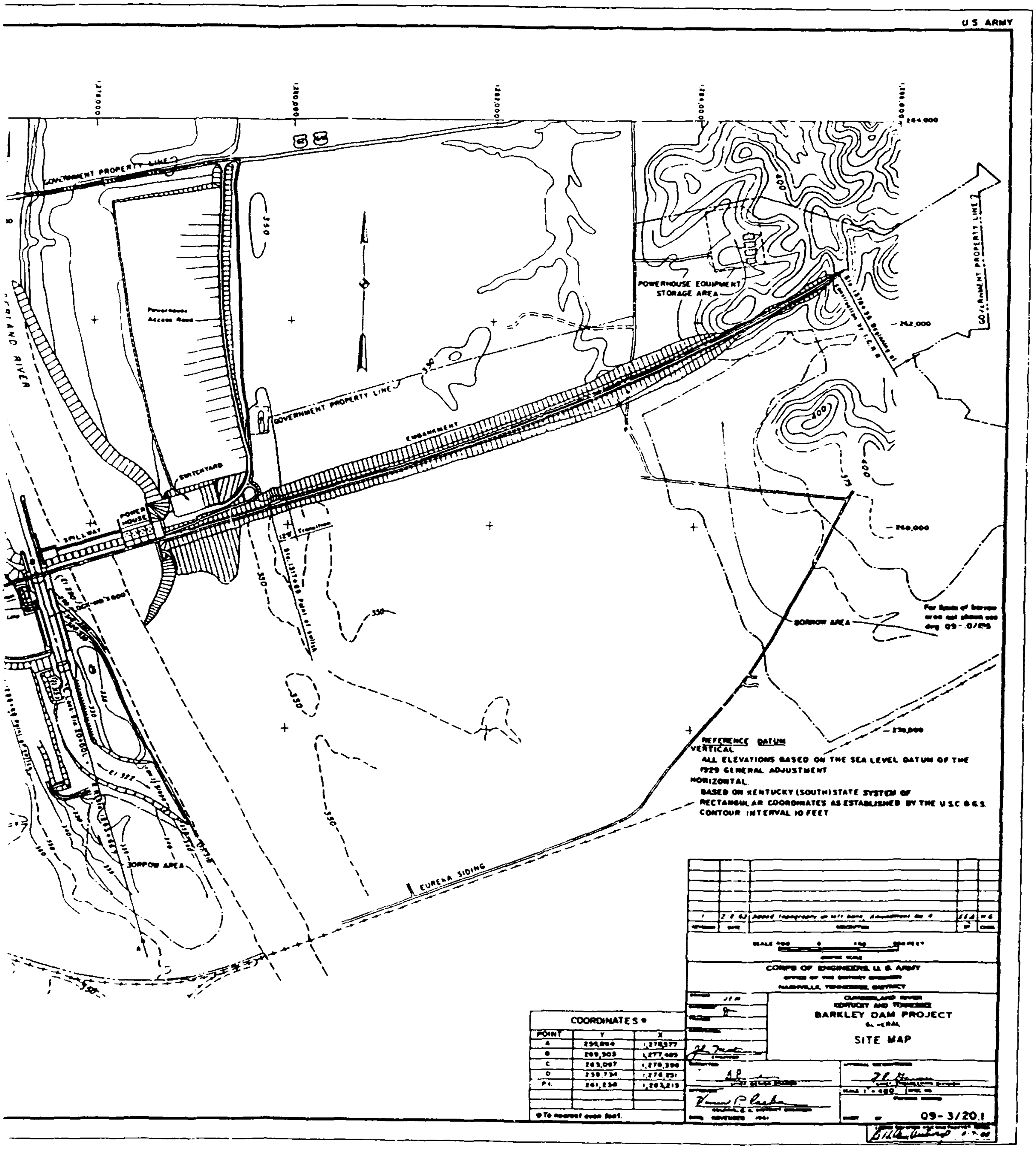


Figure 1. Plan view of the Barkley Lake and Dam Pr



n view of the Barkley Lake and Dam Project.

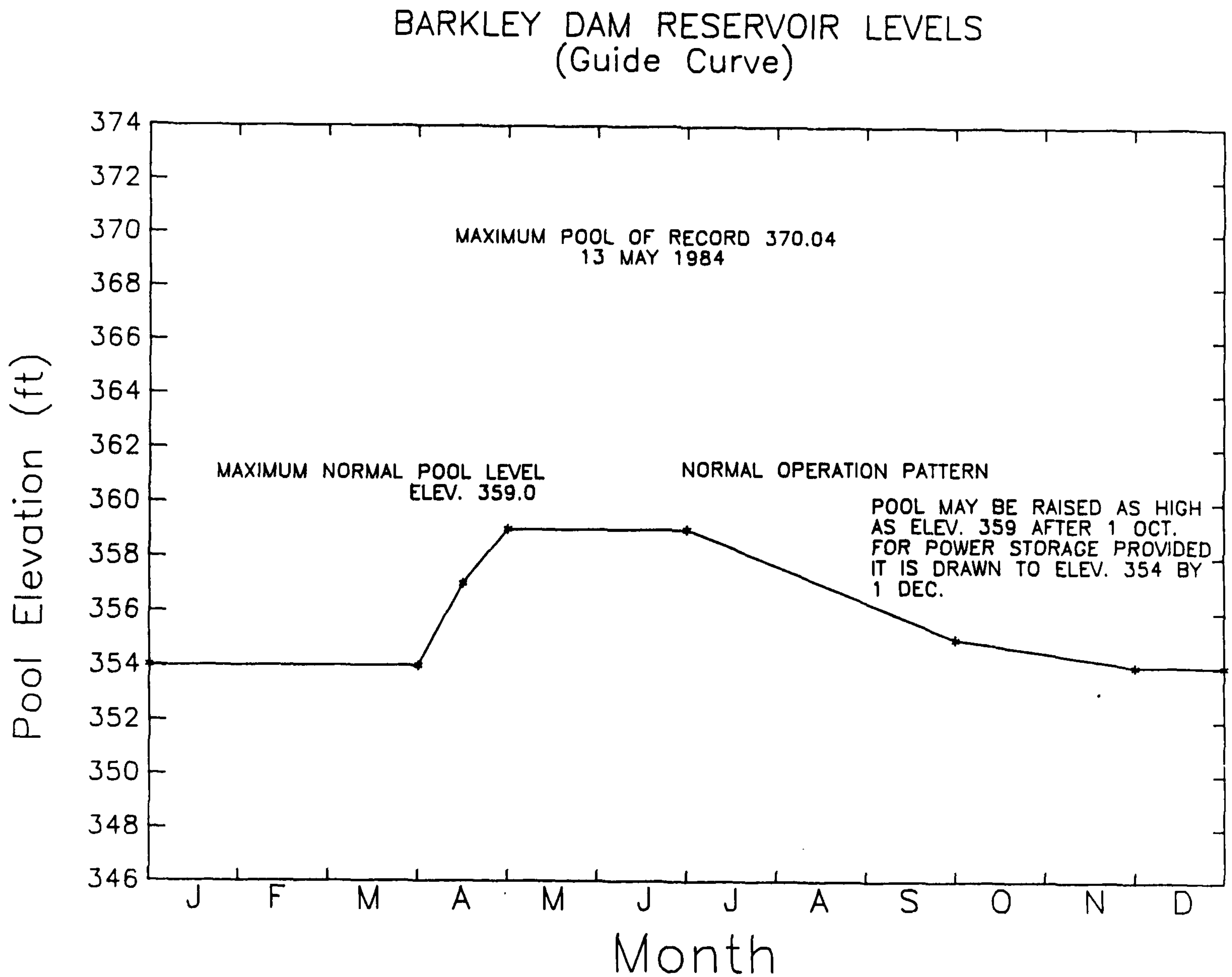


Figure 2. Guide curve for reservoir levels of Barkley Dam.

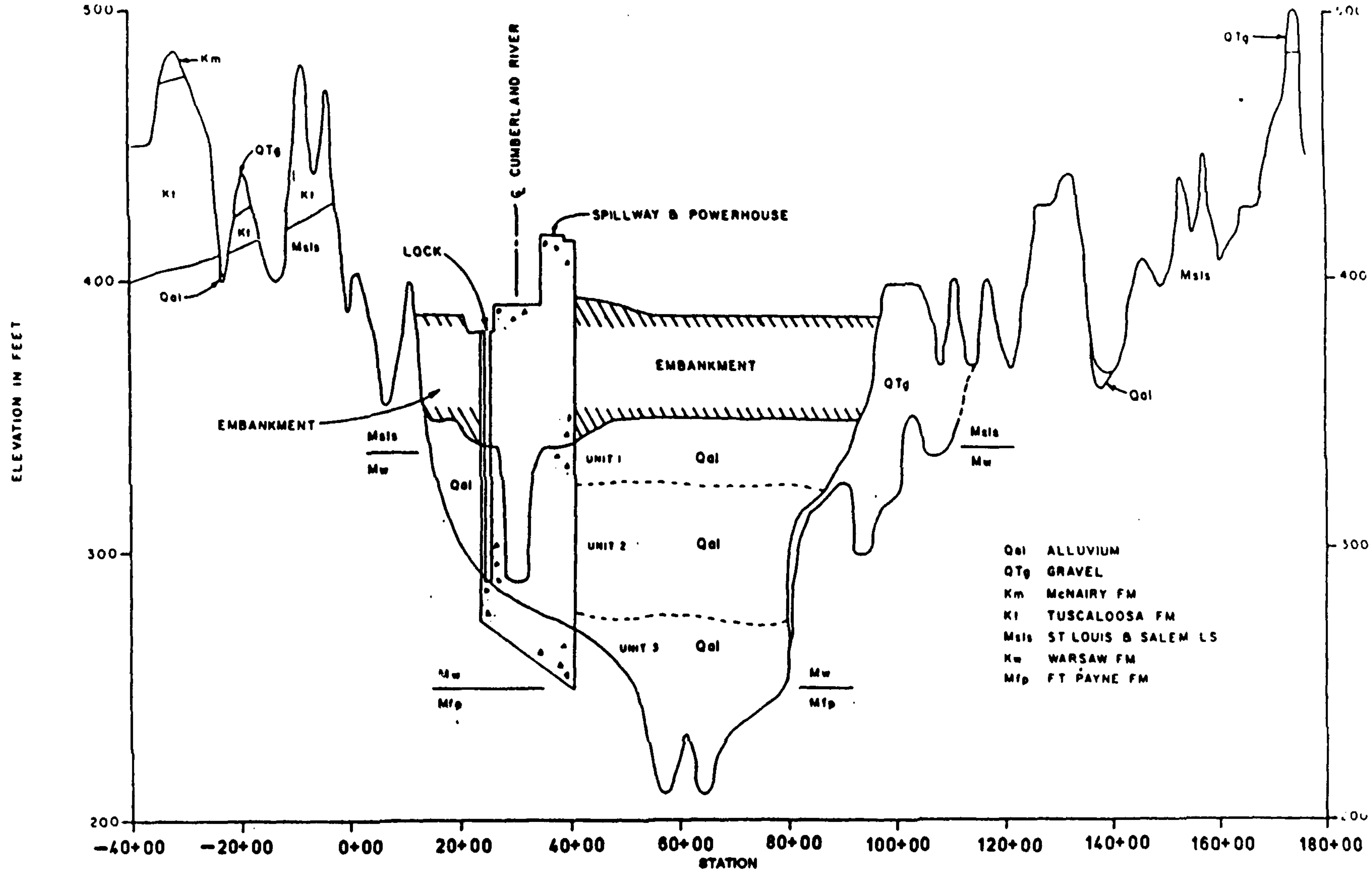


Figure 3. Generalized geologic section approximately along centerline of dam.

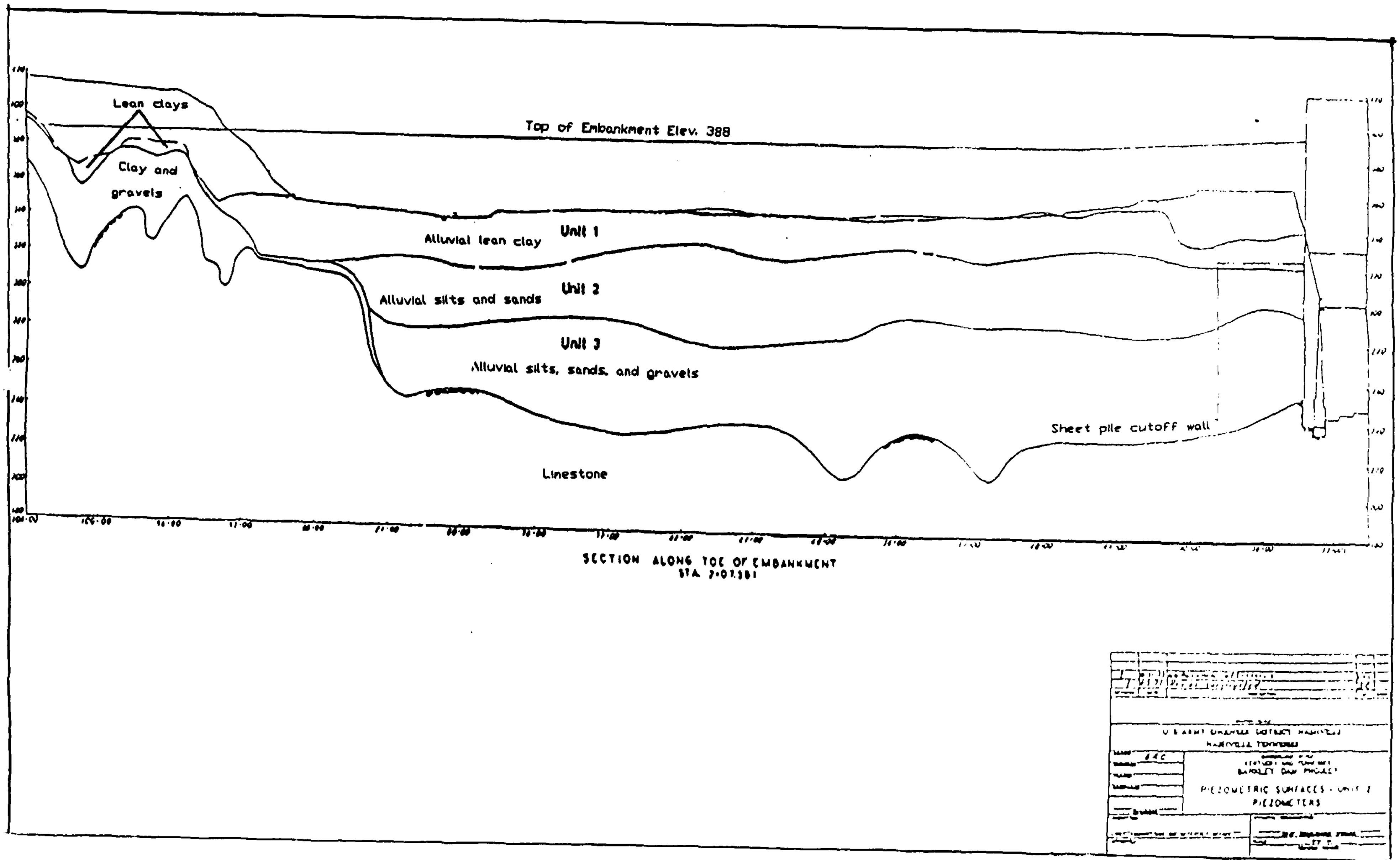


Figure 4. Longitudinal cross section along the toe of the dam showing the three major foundation units.

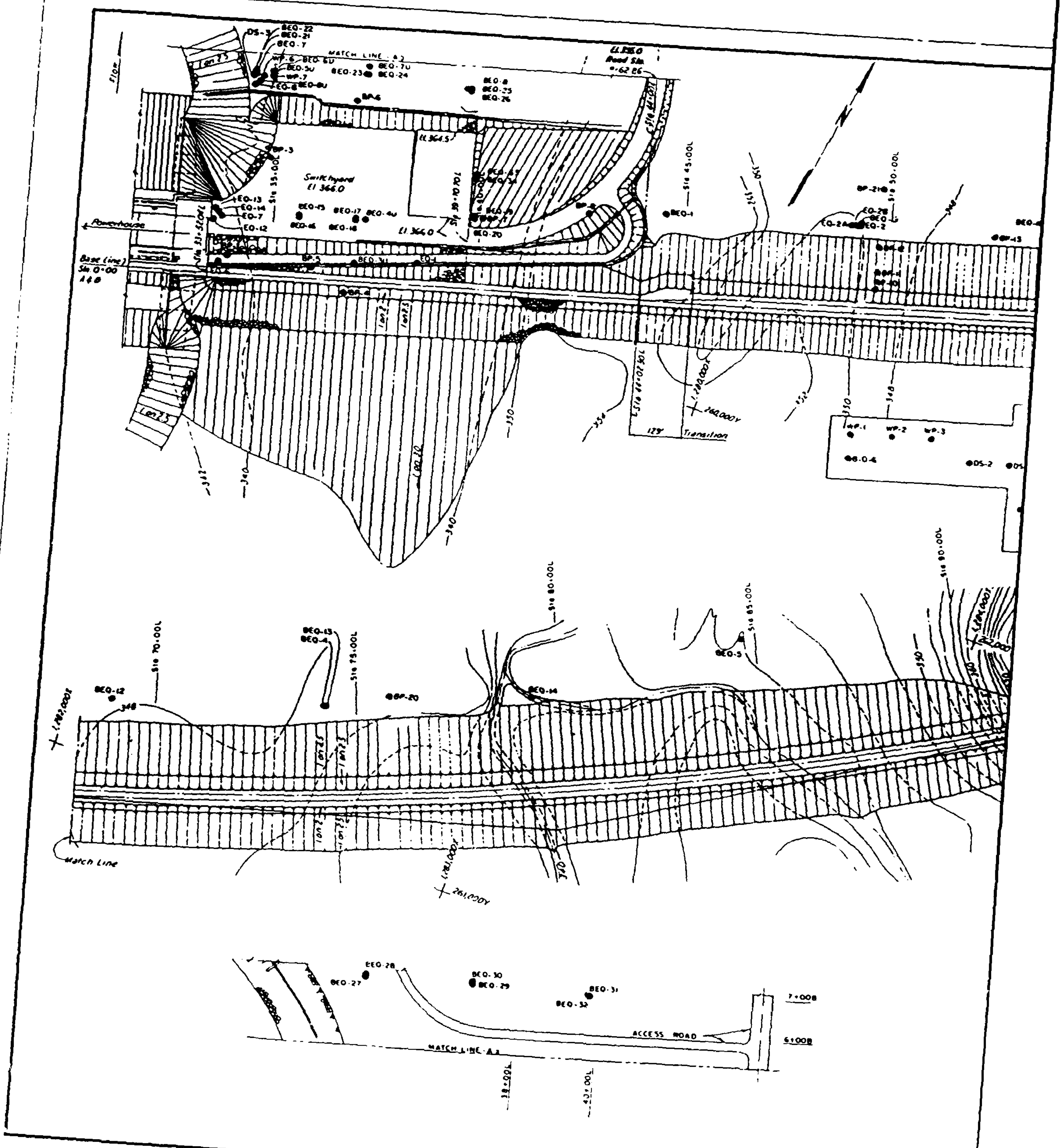
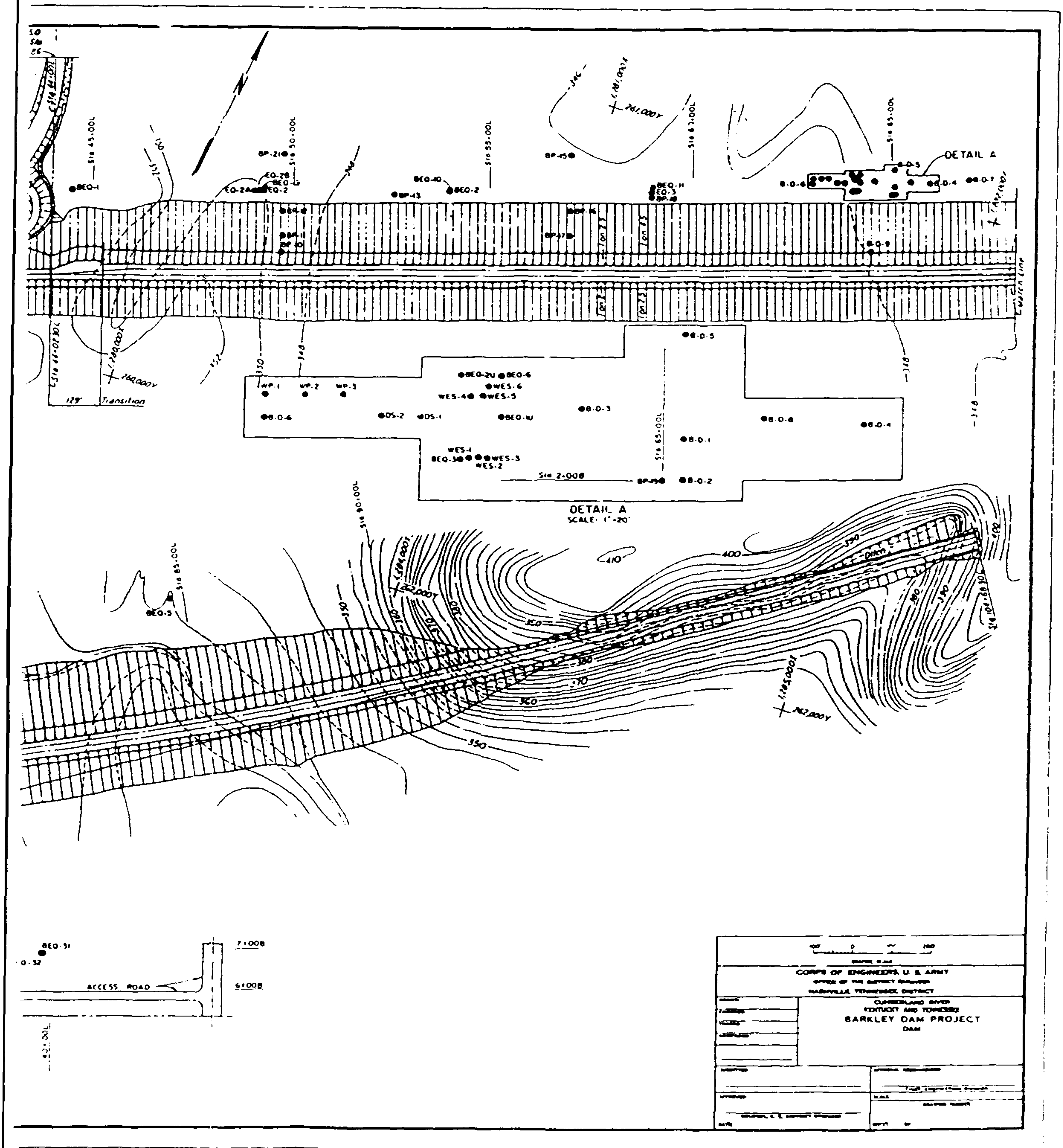


Figure 5. Plan view of Barkley Dam showing undisturbed borings and SPT



5. Plan view of Barkley Dam showing the locations of undisturbed borings and SPT soundings.

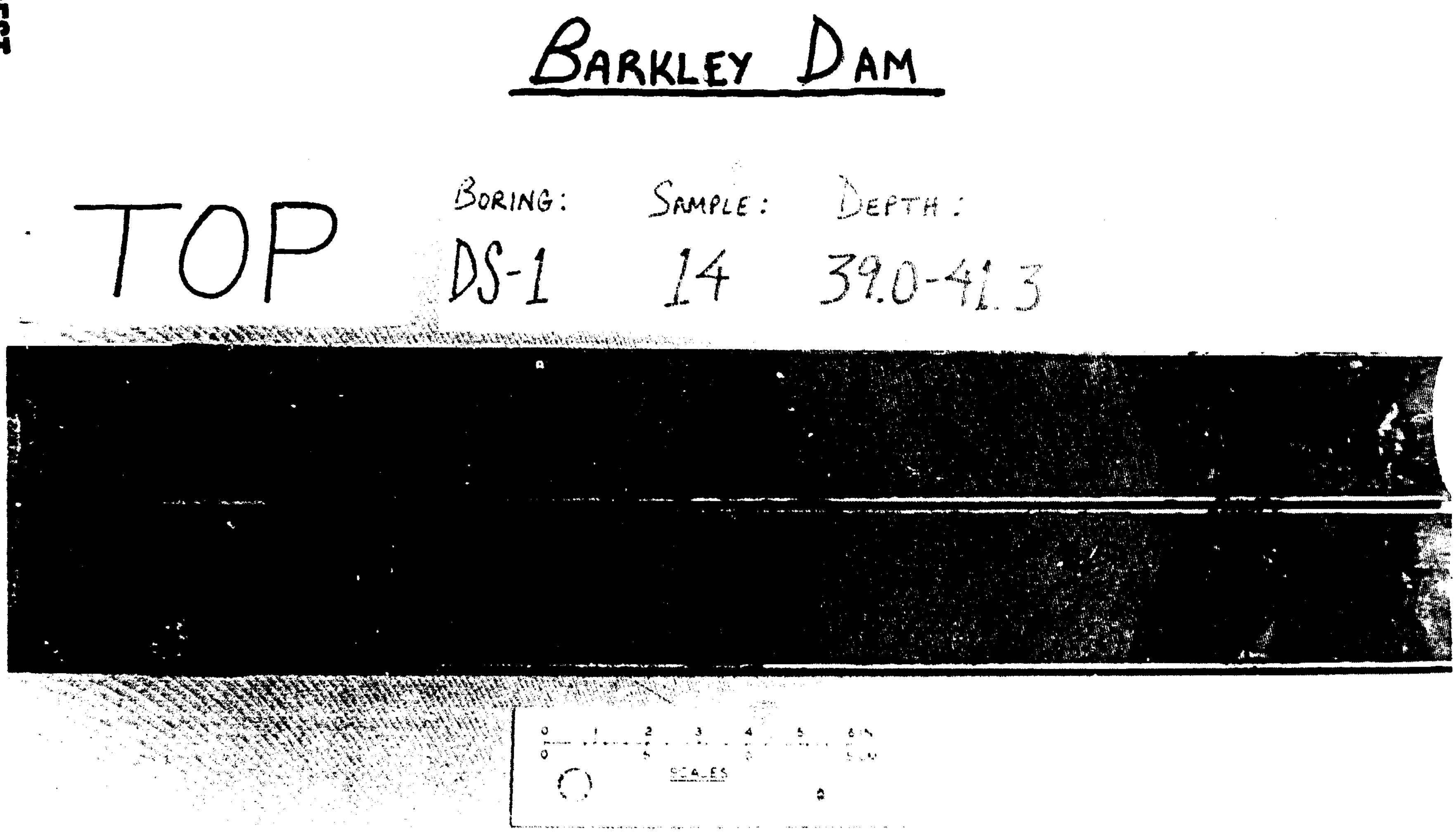


Figure 6. Photograph of typical undisturbed sample from Unit 2.

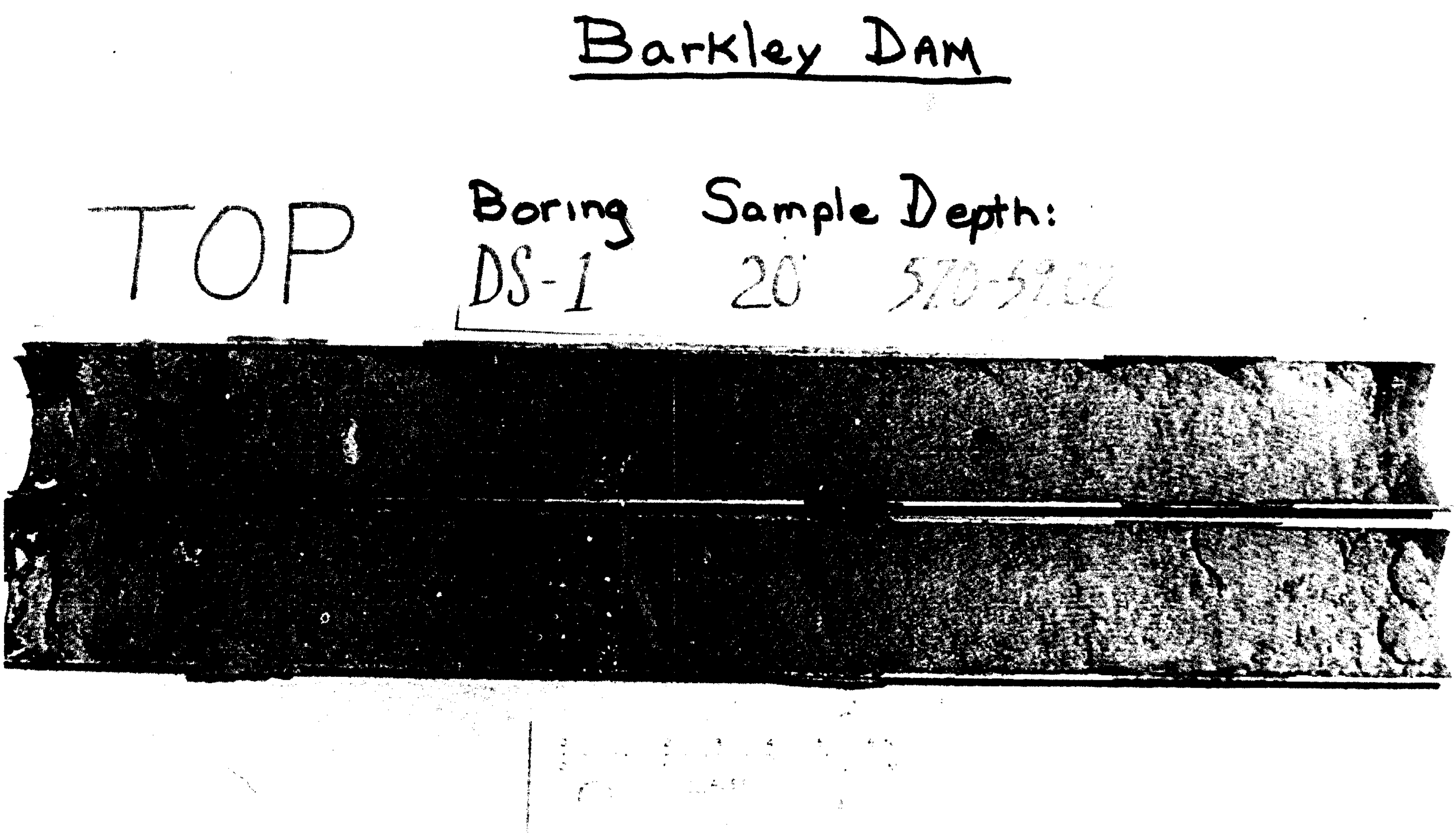


Figure 7. Photograph of typical undisturbed sample from Unit 3.

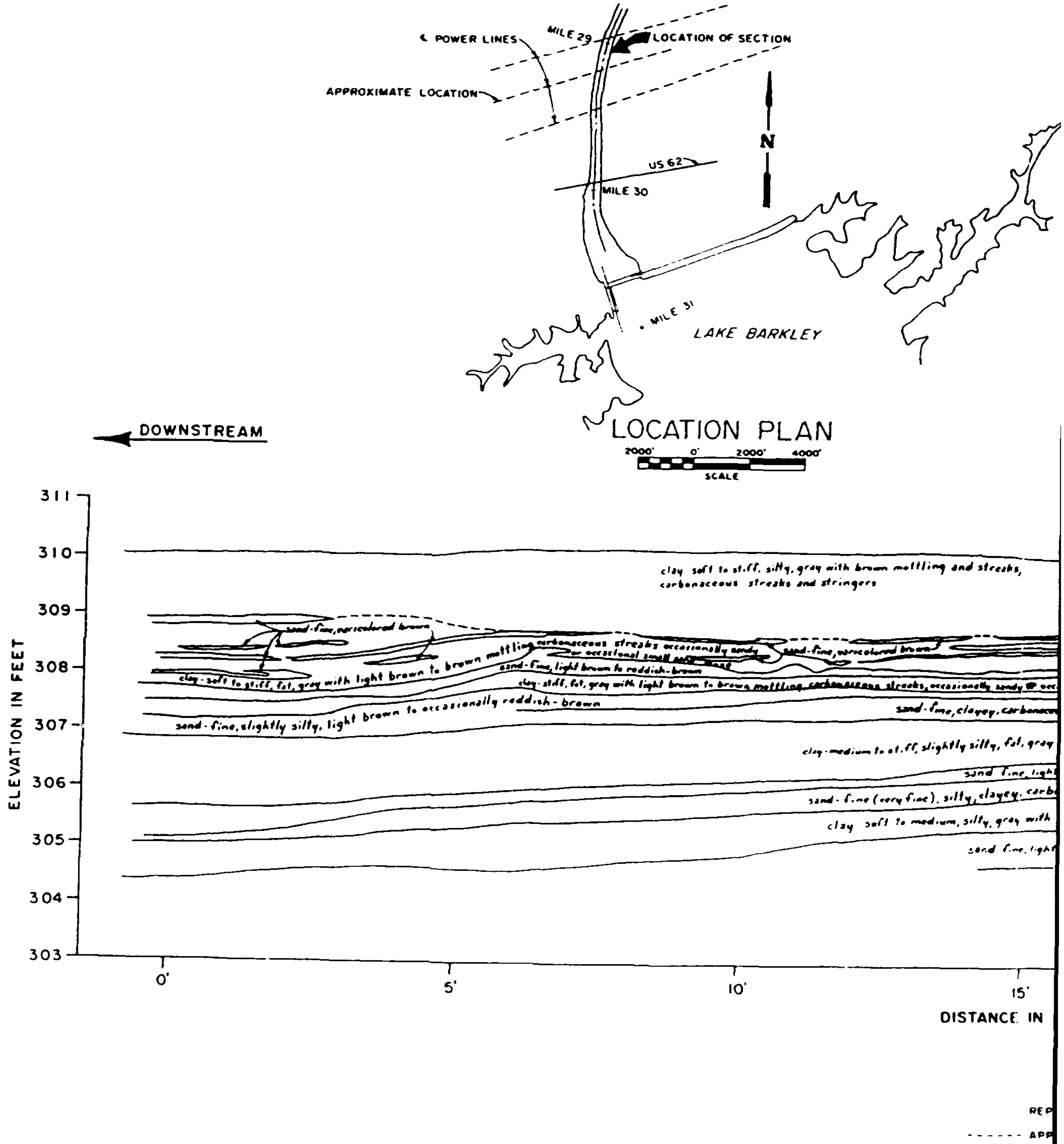
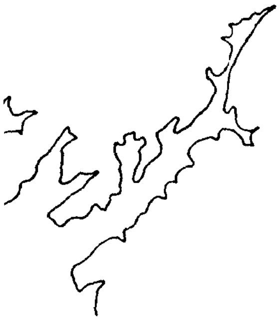
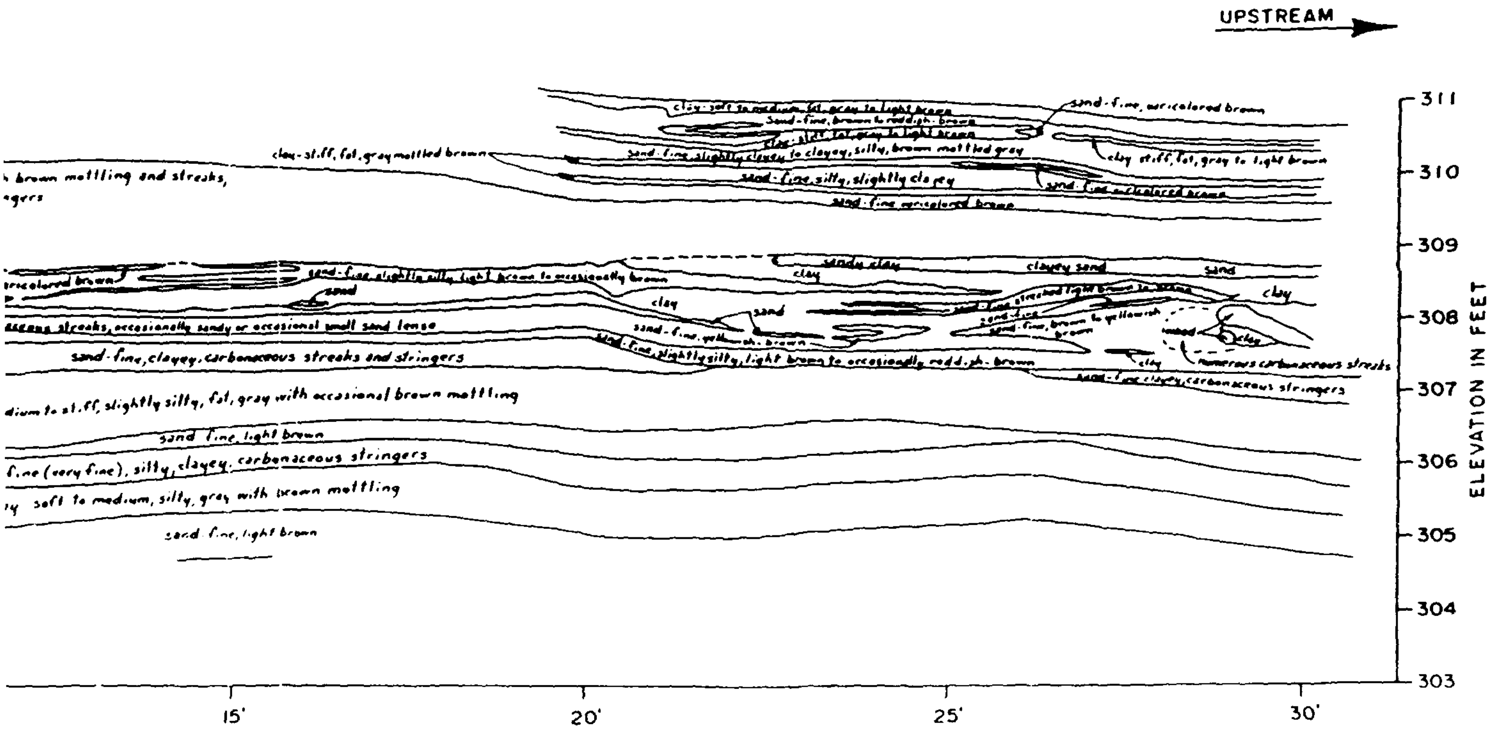


Figure 8. Geologic cross section of the exposu
streambank excavation downstream of





LEGEND

REPRESENTS GRADATIONAL CHANGE

------ APPROXIMATE CONTACT

NOTES

1 THE ABOVE DESCRIPTIONS ARE FROM VISUAL CLASSIFICATIONS
2 THE LOCATION OF THE ABOVE SECTION IS ON THE RIGHT BANK ABOUT 1-1/2 MILES DOWNSTREAM OF BARKLEY DAM SEE LOCATION PLAN
3 ALTHOUGH NOT SHOWN ON ABOVE SECTION, ONE SAND BED COULD BE TRACED FOR APPROXIMATELY 150 FEET BEFORE IT WAS LOST
4 MAPPING DONE 31 OCT - 1 NOV 1983

BARKLEY DAM SEISMIC STUDIES

ion of the exposure of the
on downstream of the dam.

Figure [illegible]

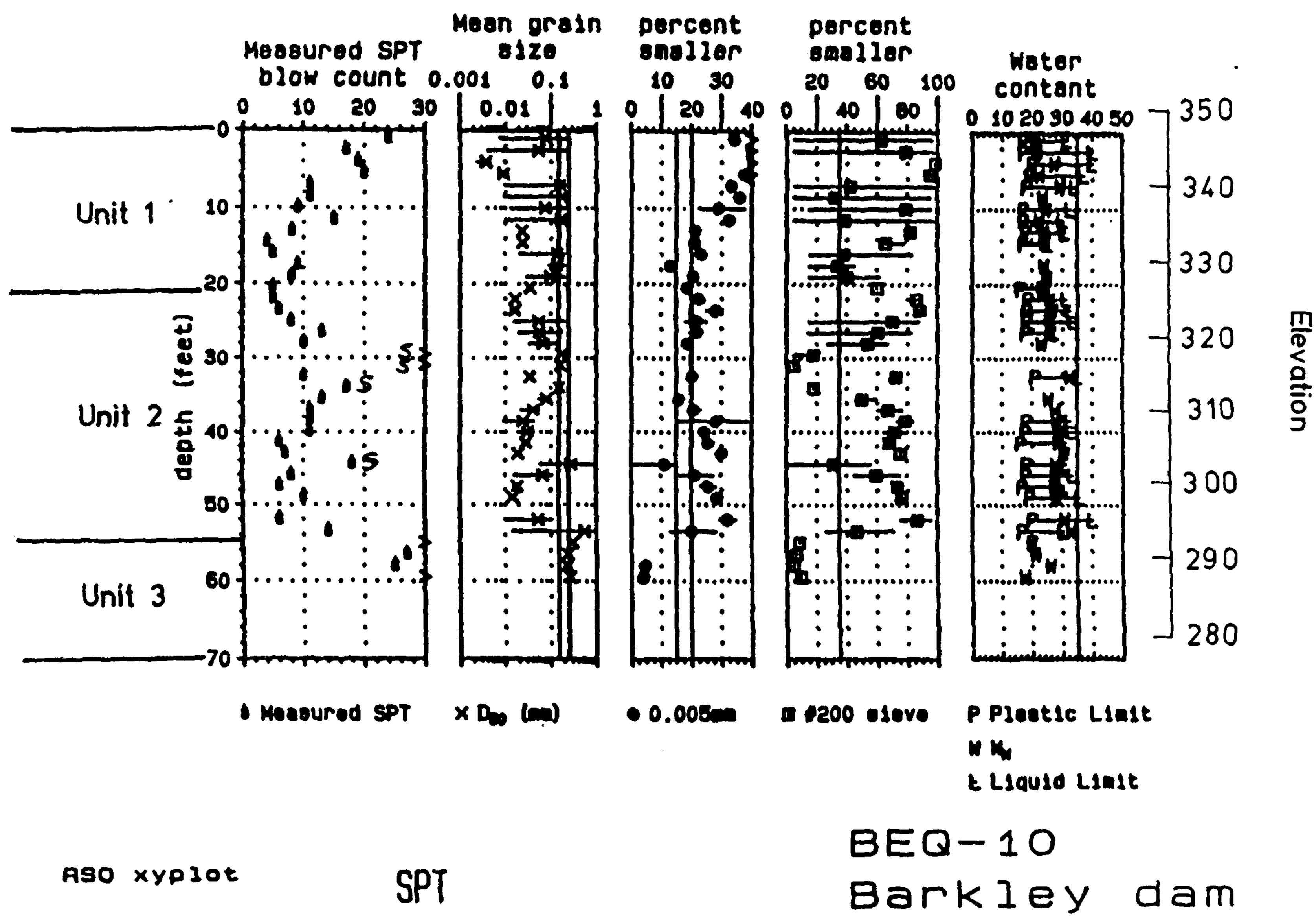


Figure 10. SPT data from SPT boring BEQ-10.

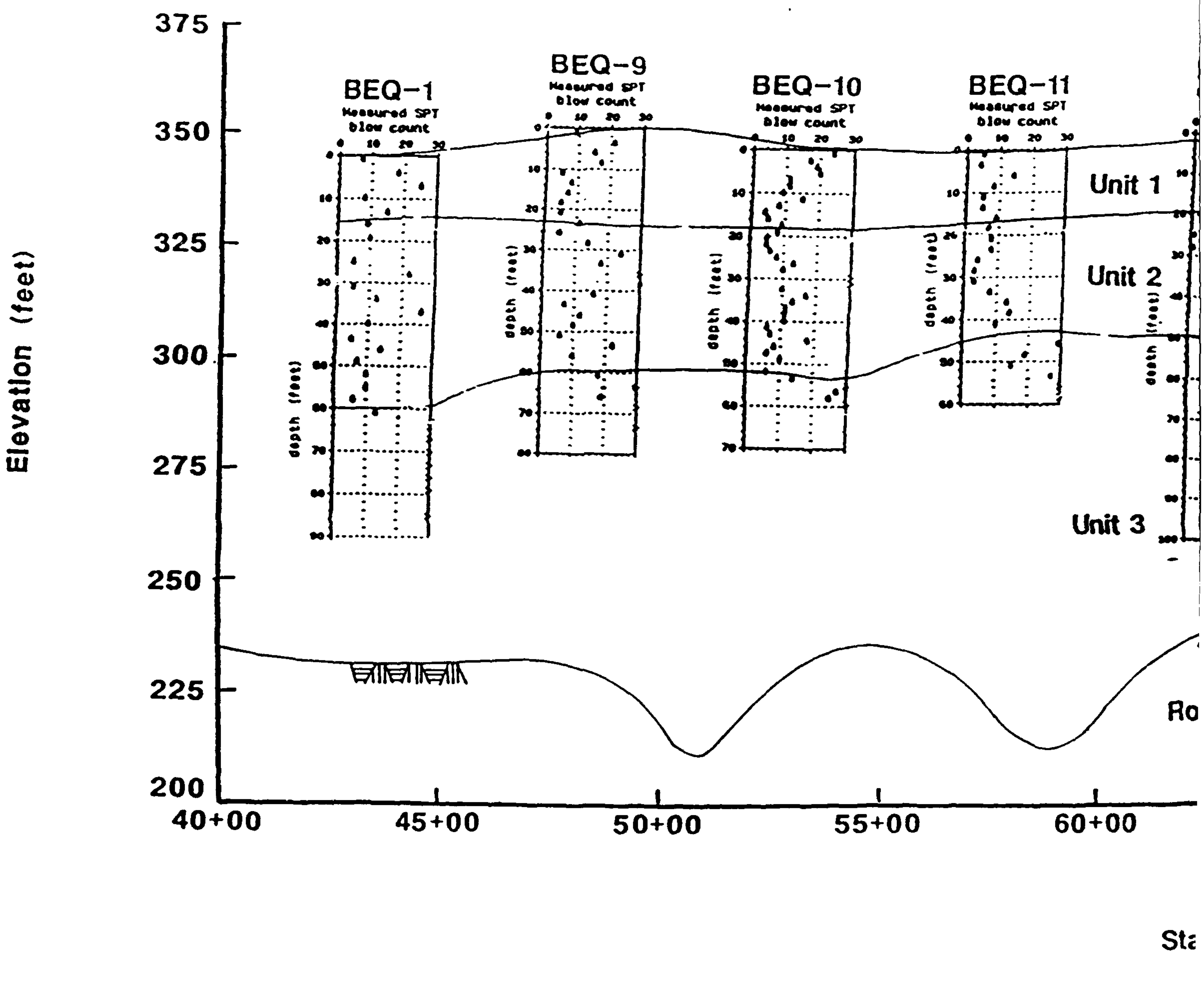
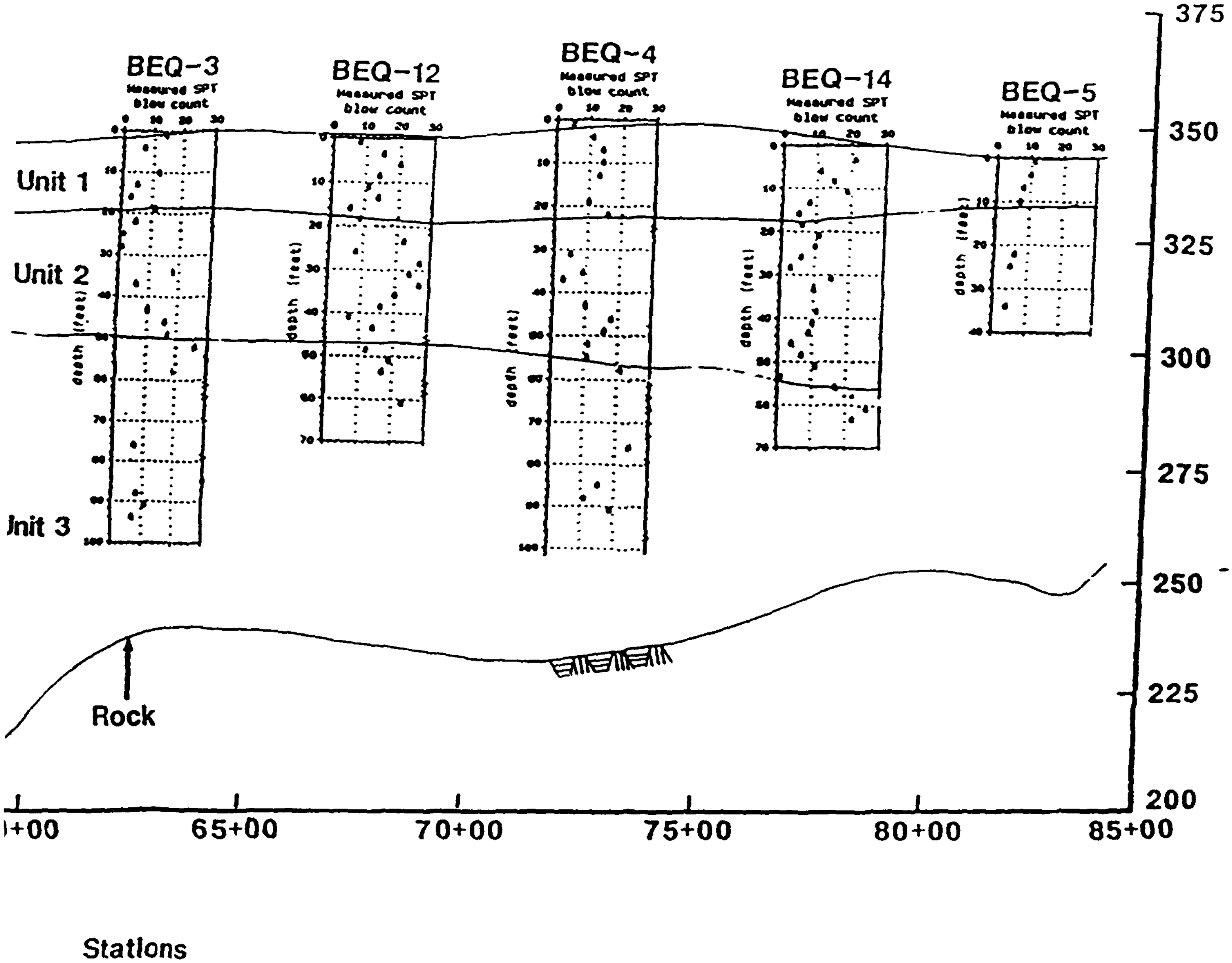


Figure 11. SPT cross section along



:ion along the toe of the main embankment.

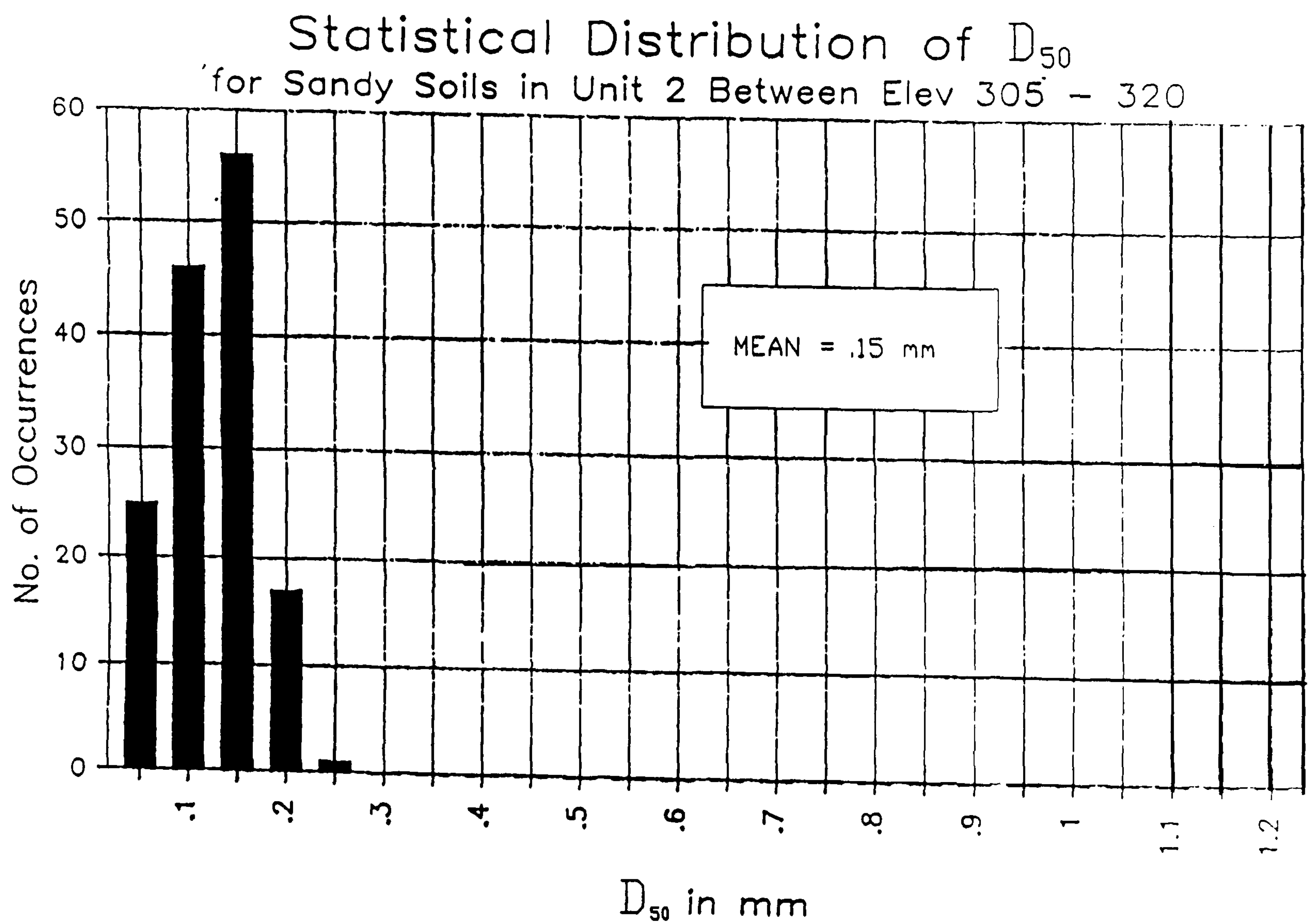


Figure 12. Histogram of D_{50} for Unit 2 sands.

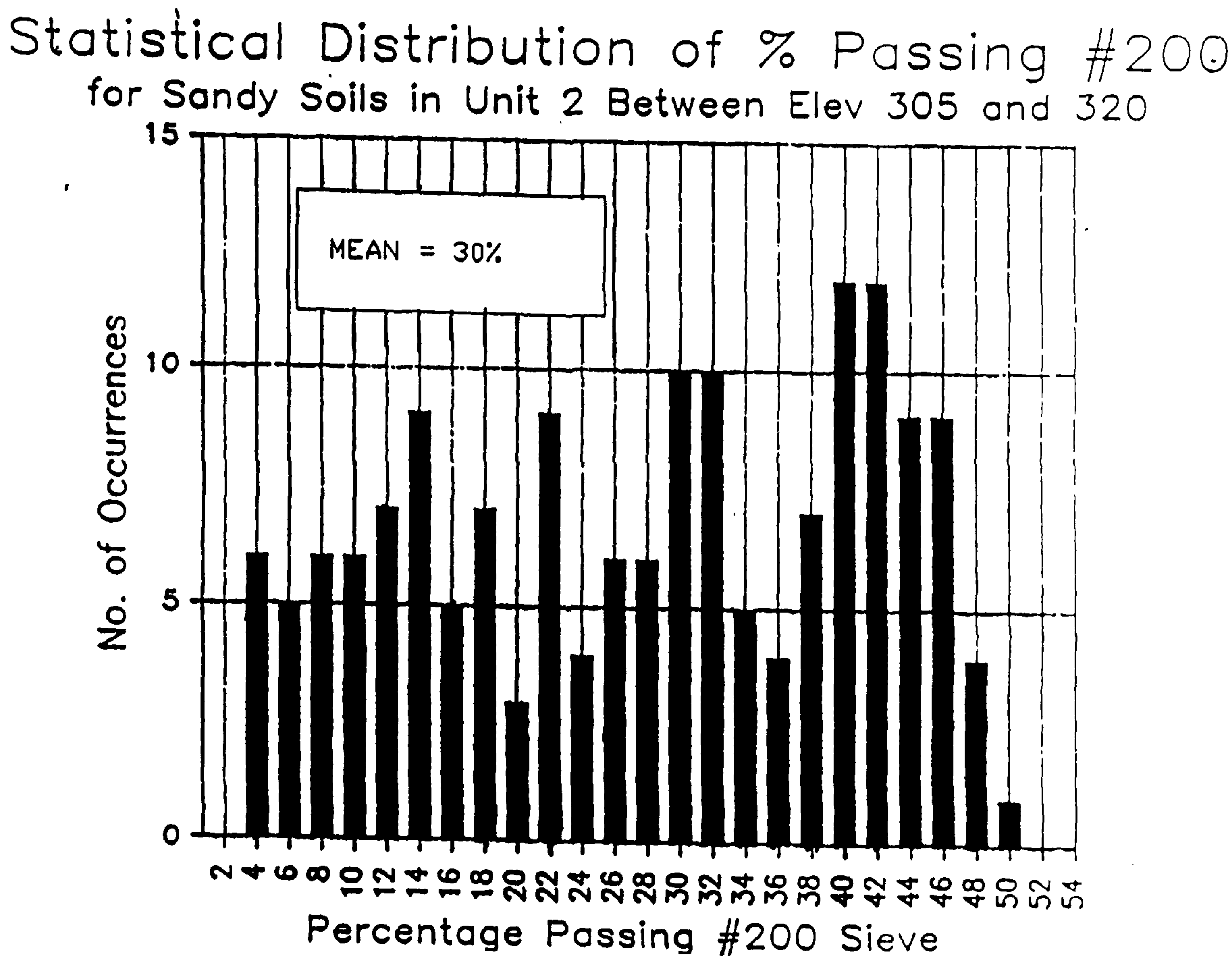


Figure 13. Histogram of percentage passing the #200 sieve for Unit 2 sands.

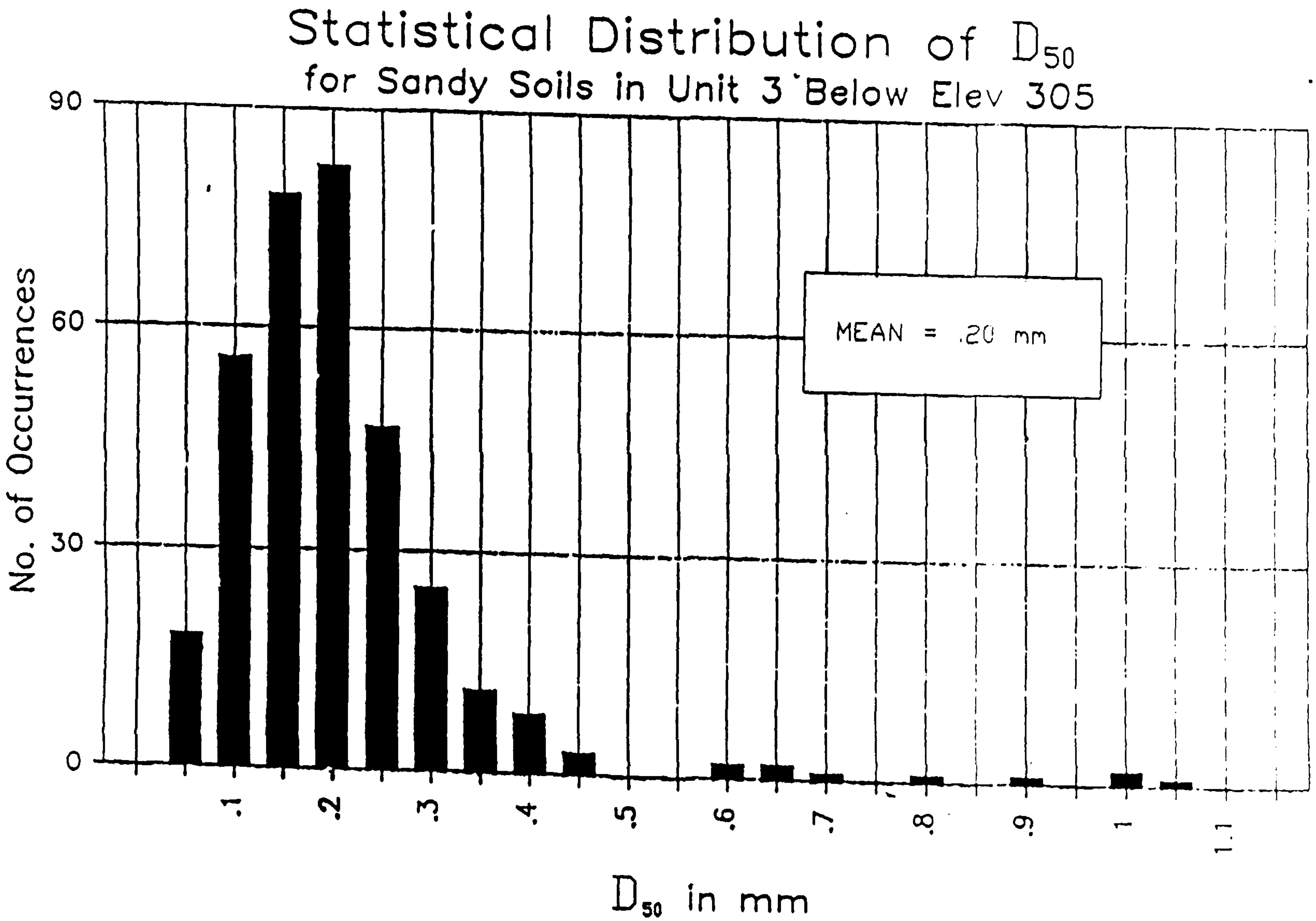


Figure 14. Histogram of D_{50} for Unit 3 sands.

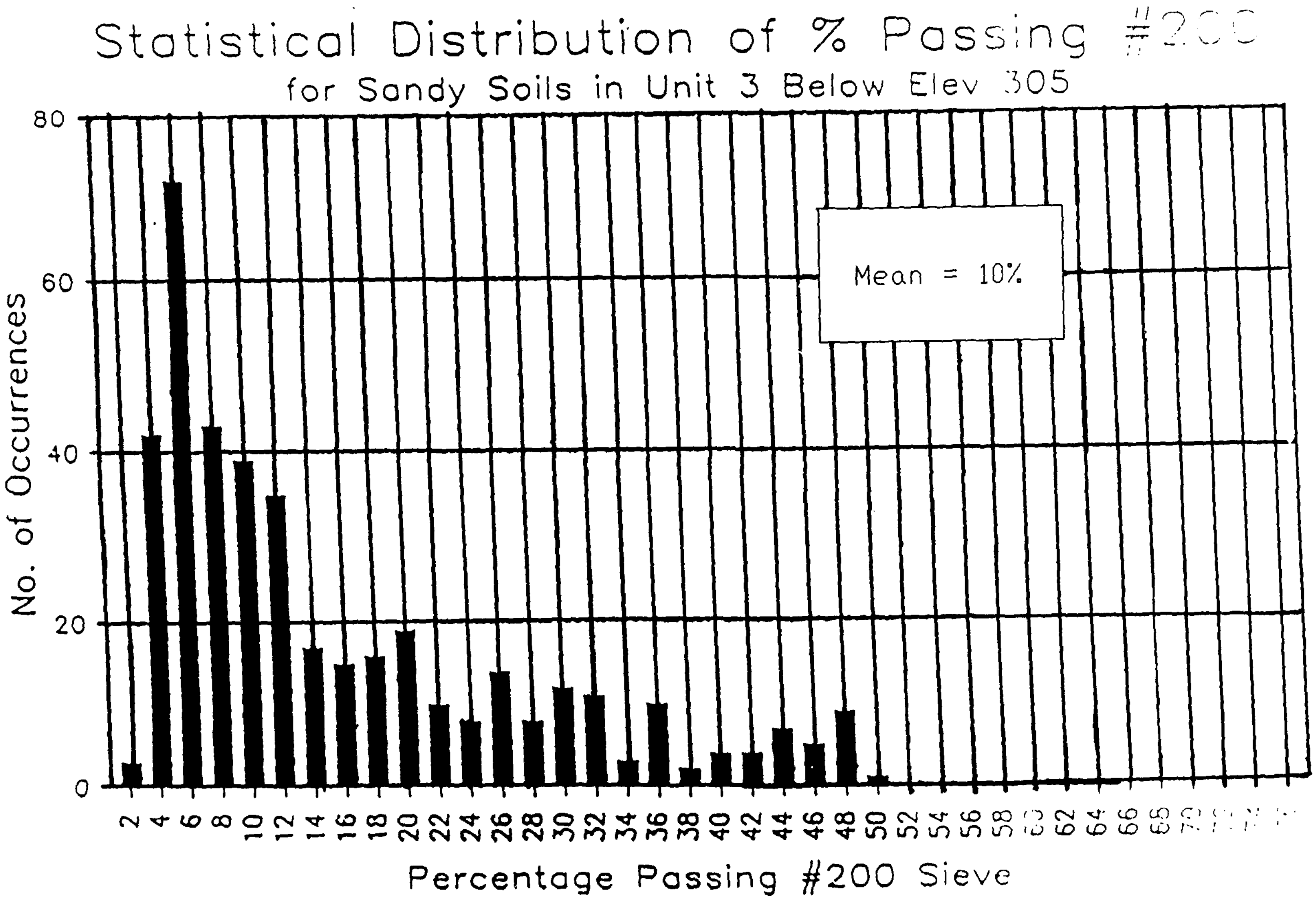


Figure 15. Histogram of the percentage passing the #200 sieve for Unit 3 sands.

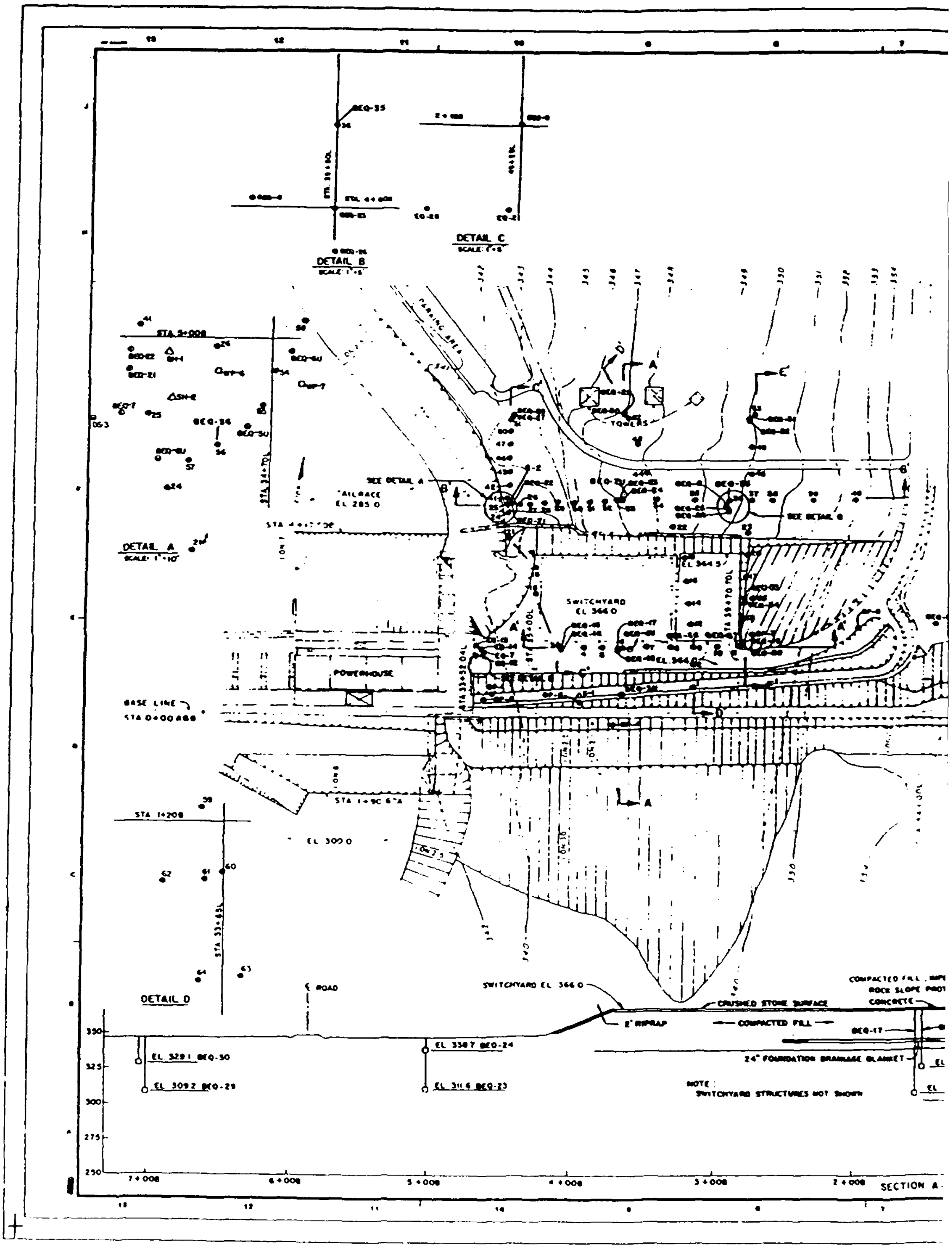
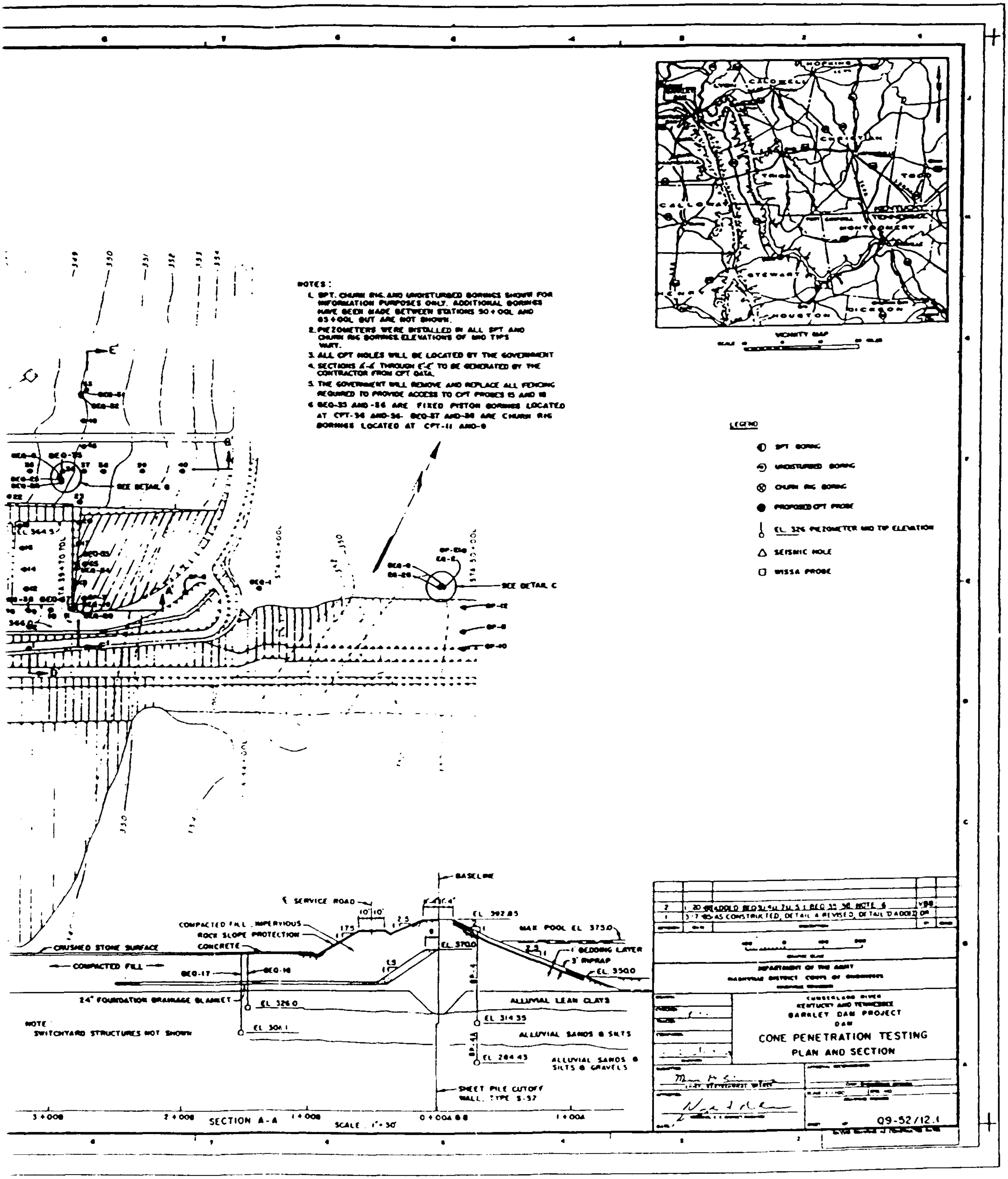


Figure 16. Plan showing the locations of the CPT sou



locations of the CPT soundings, SPT, and undisturbed borings.

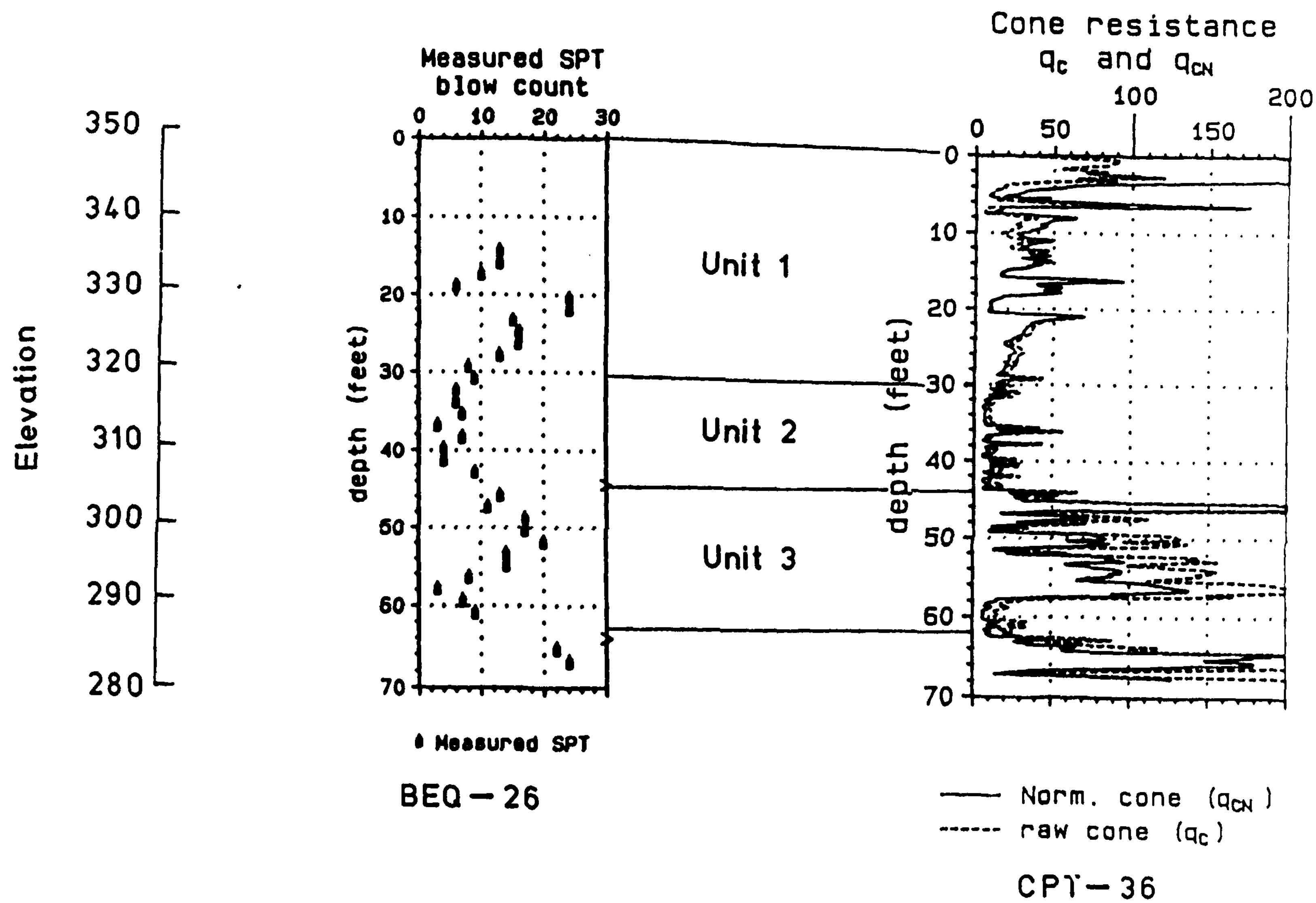


Figure 17. SPT vs. CPT comparison.

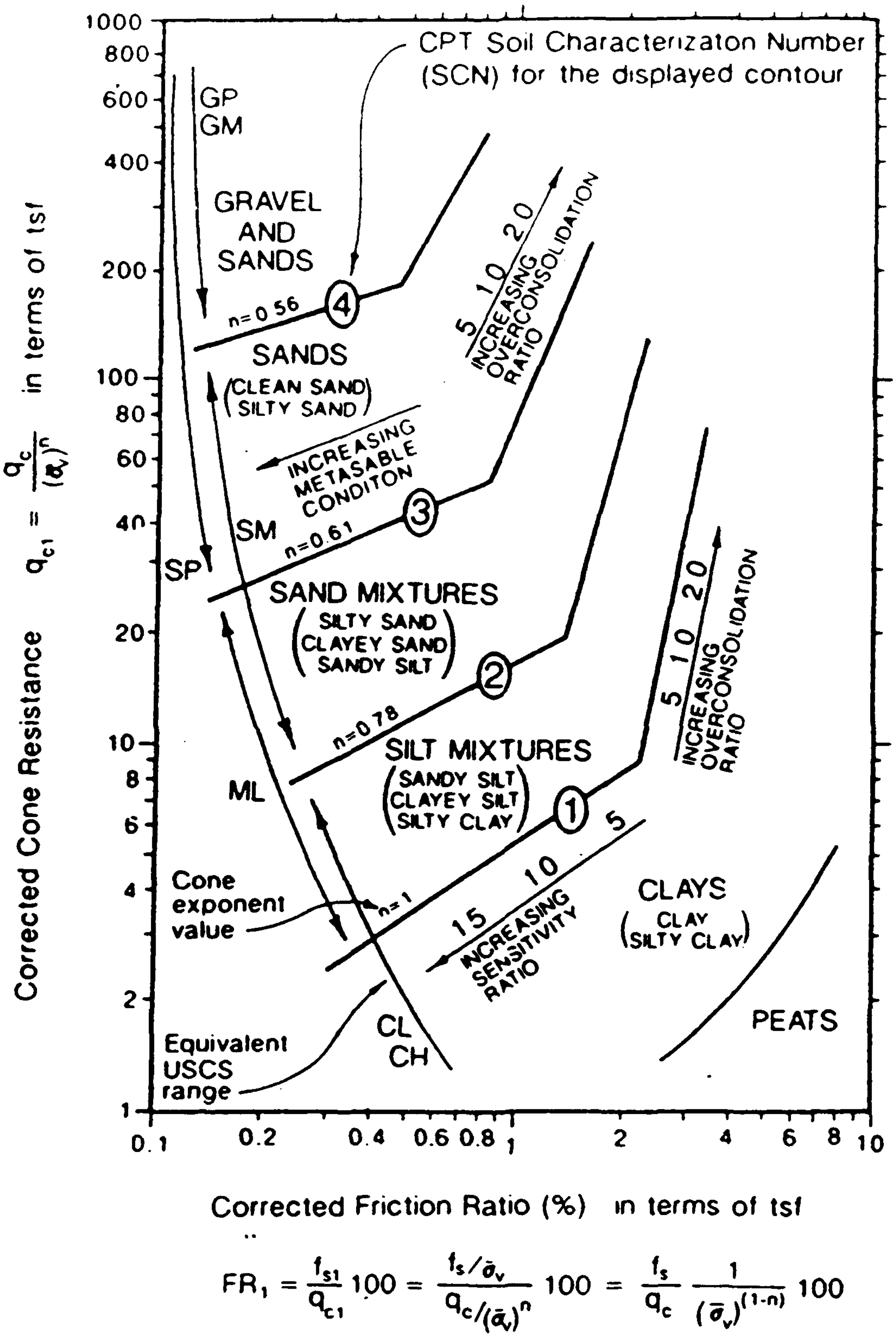


Figure 18. Soil Classification Number (Olsen, 1984).

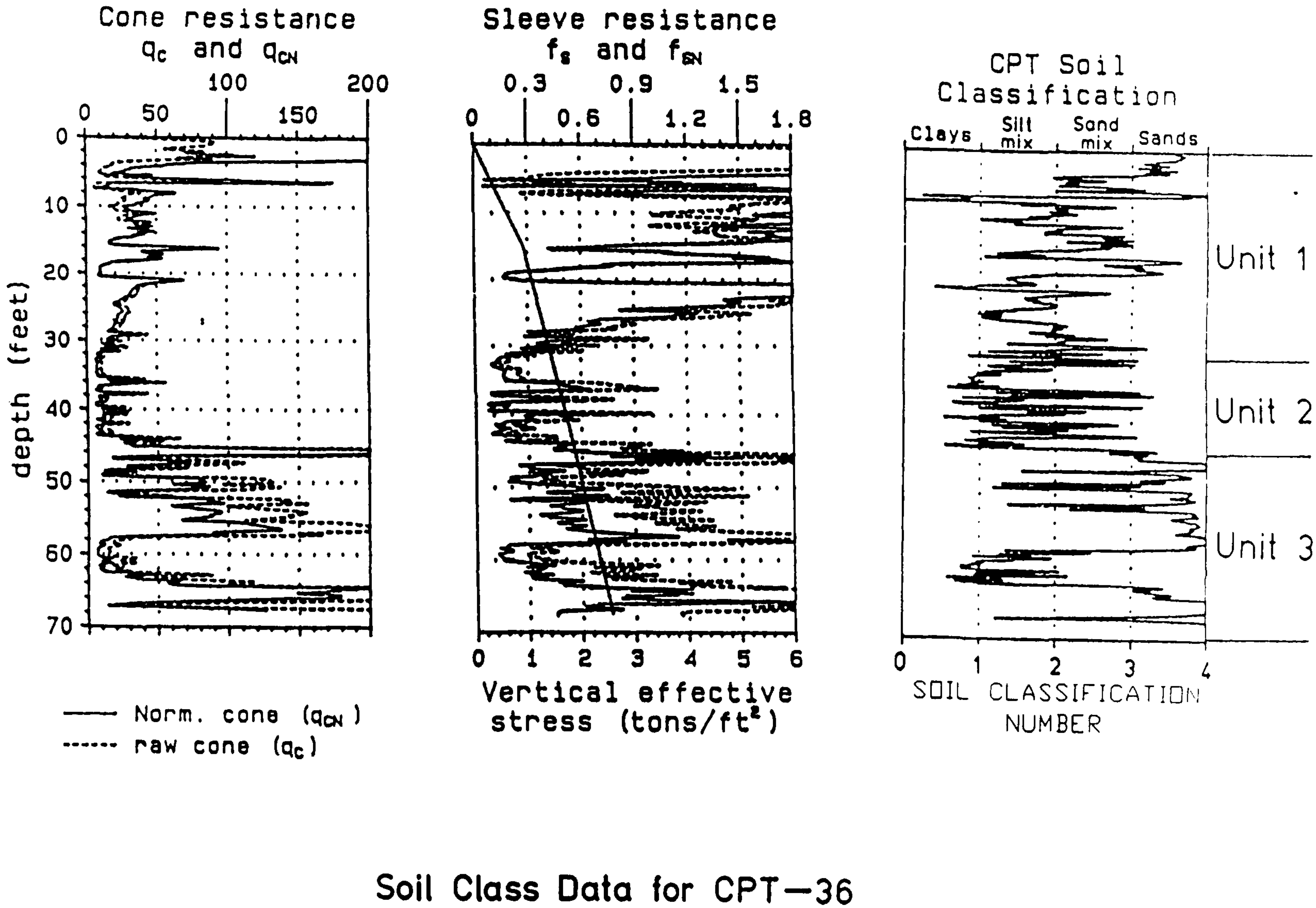


Figure 19. Typical CPT data for soil classification.

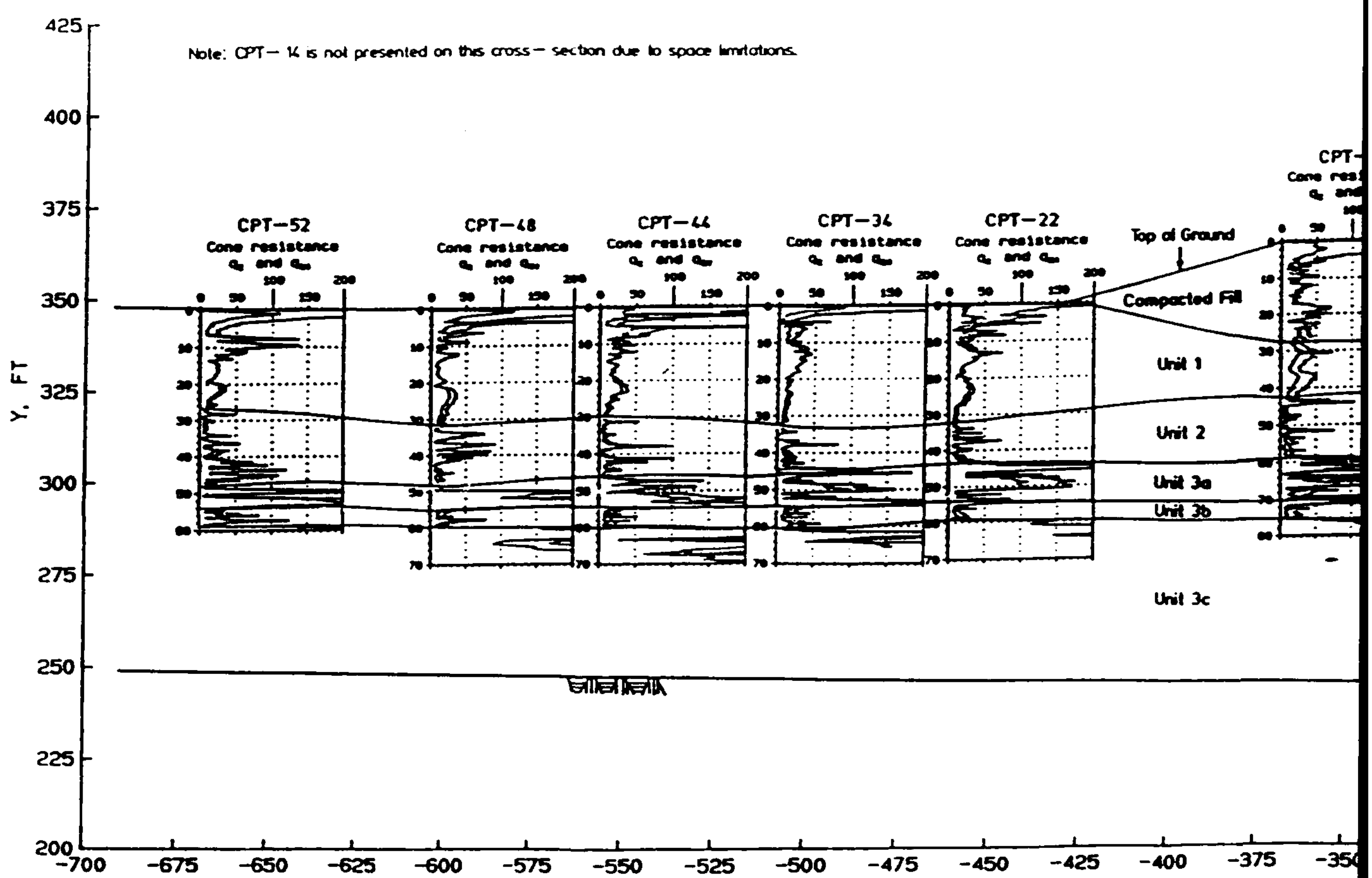
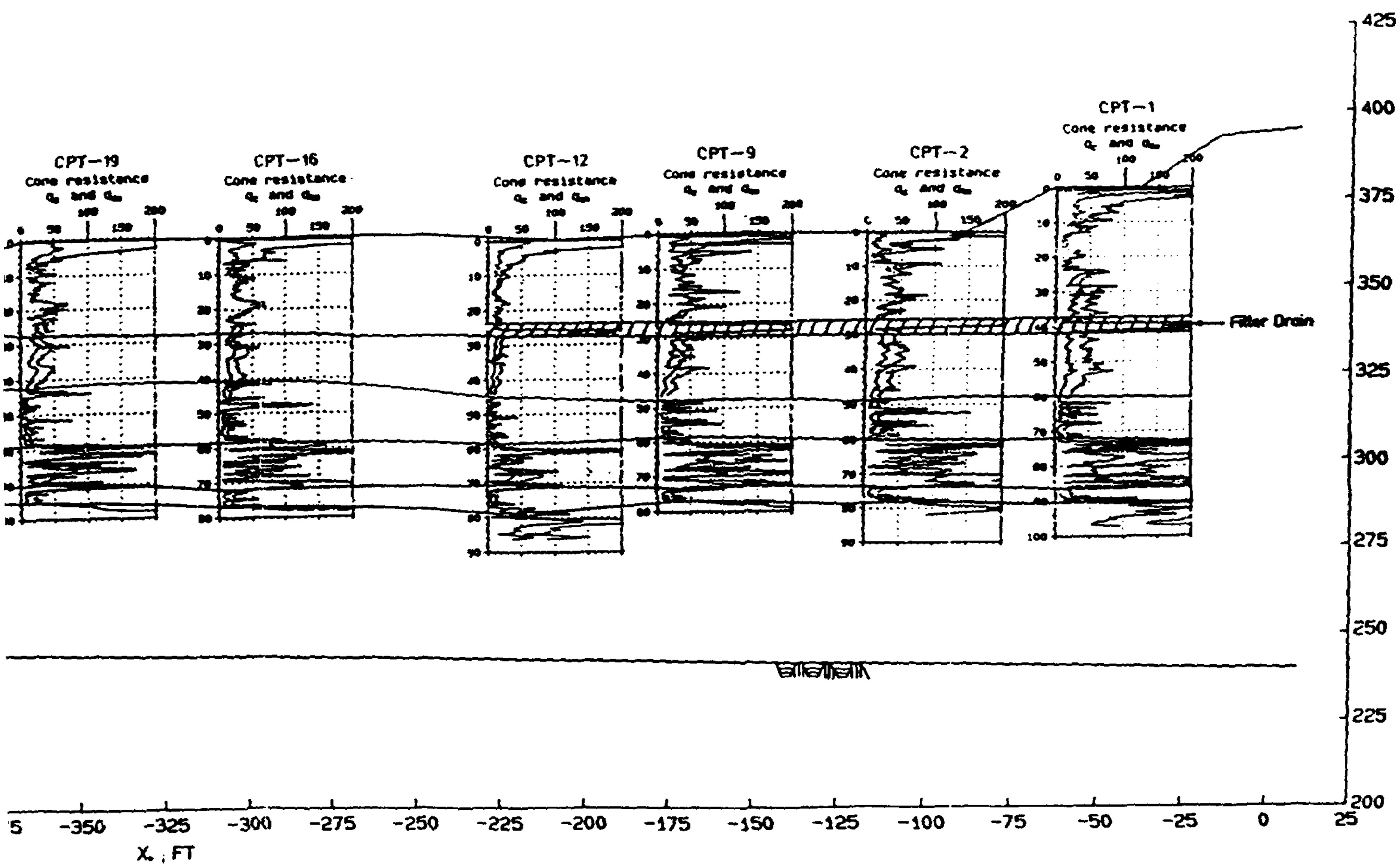


Figure 20. CPT c
(Running perpen



CPT cone resistance along section D'-D'
perpendicular to the axis of the dam).

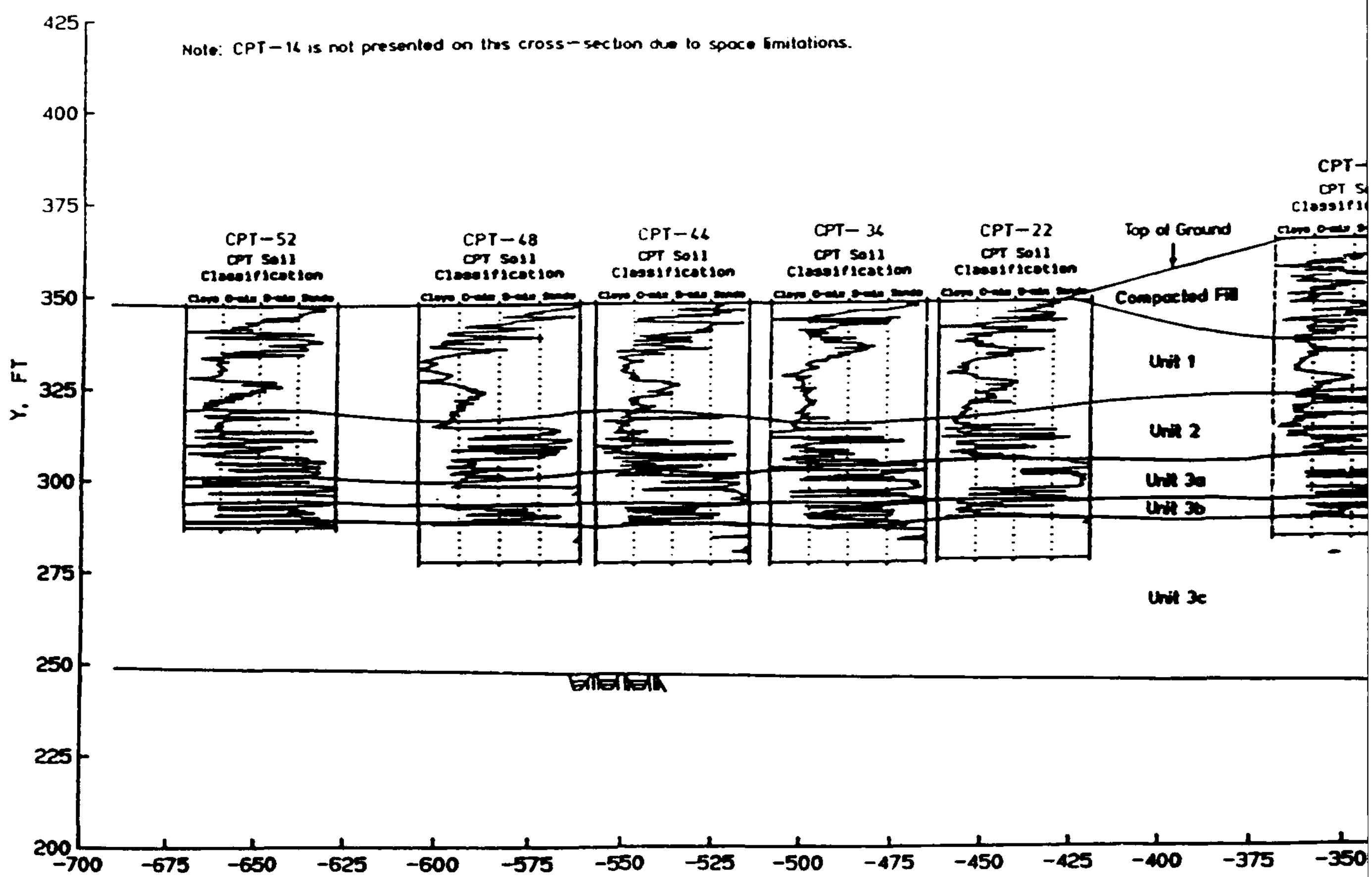
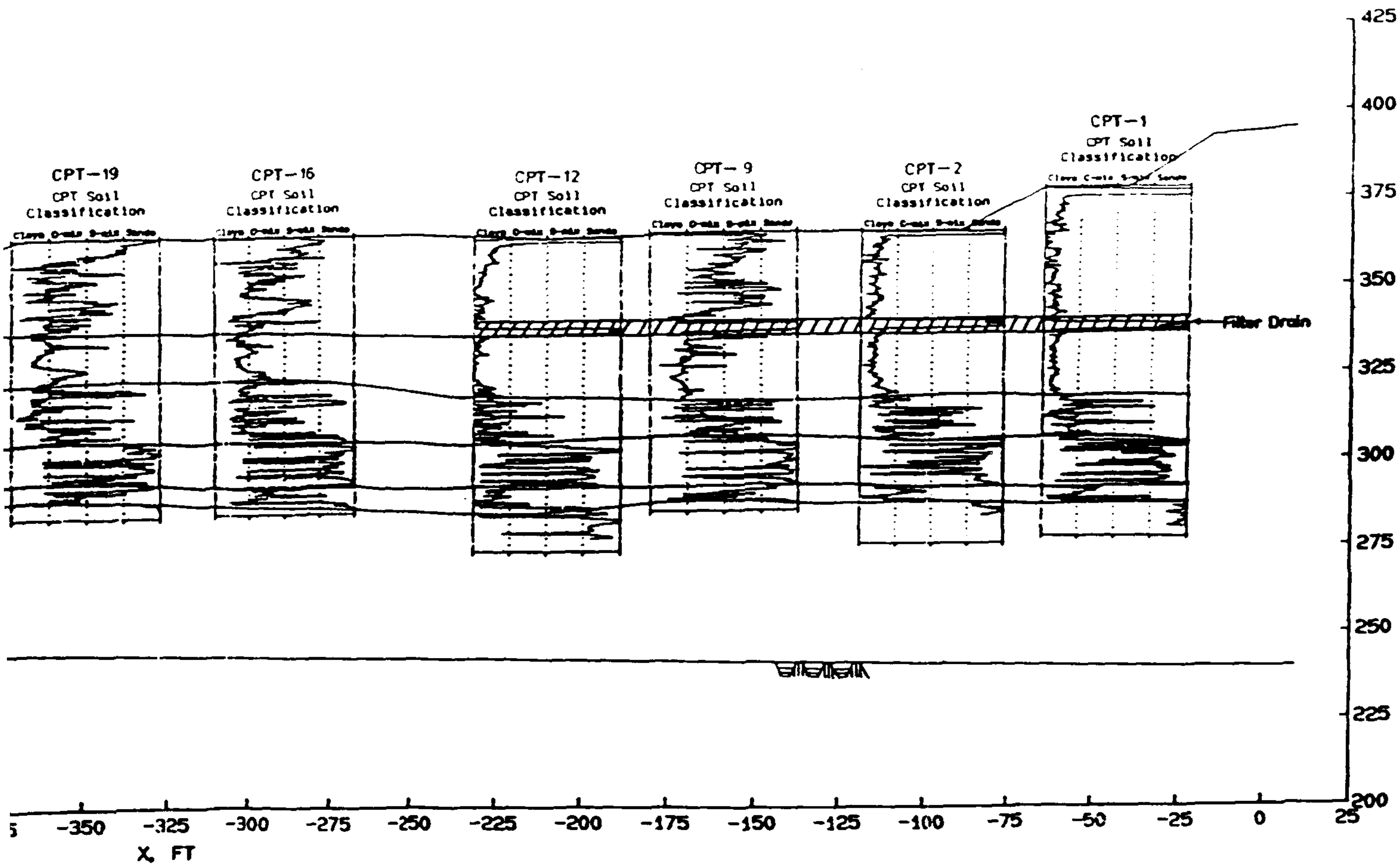


Figure 21. CPT soil classi
(Running perpendicular



classifications along section D'-D'
dicular to the axis of the dam).

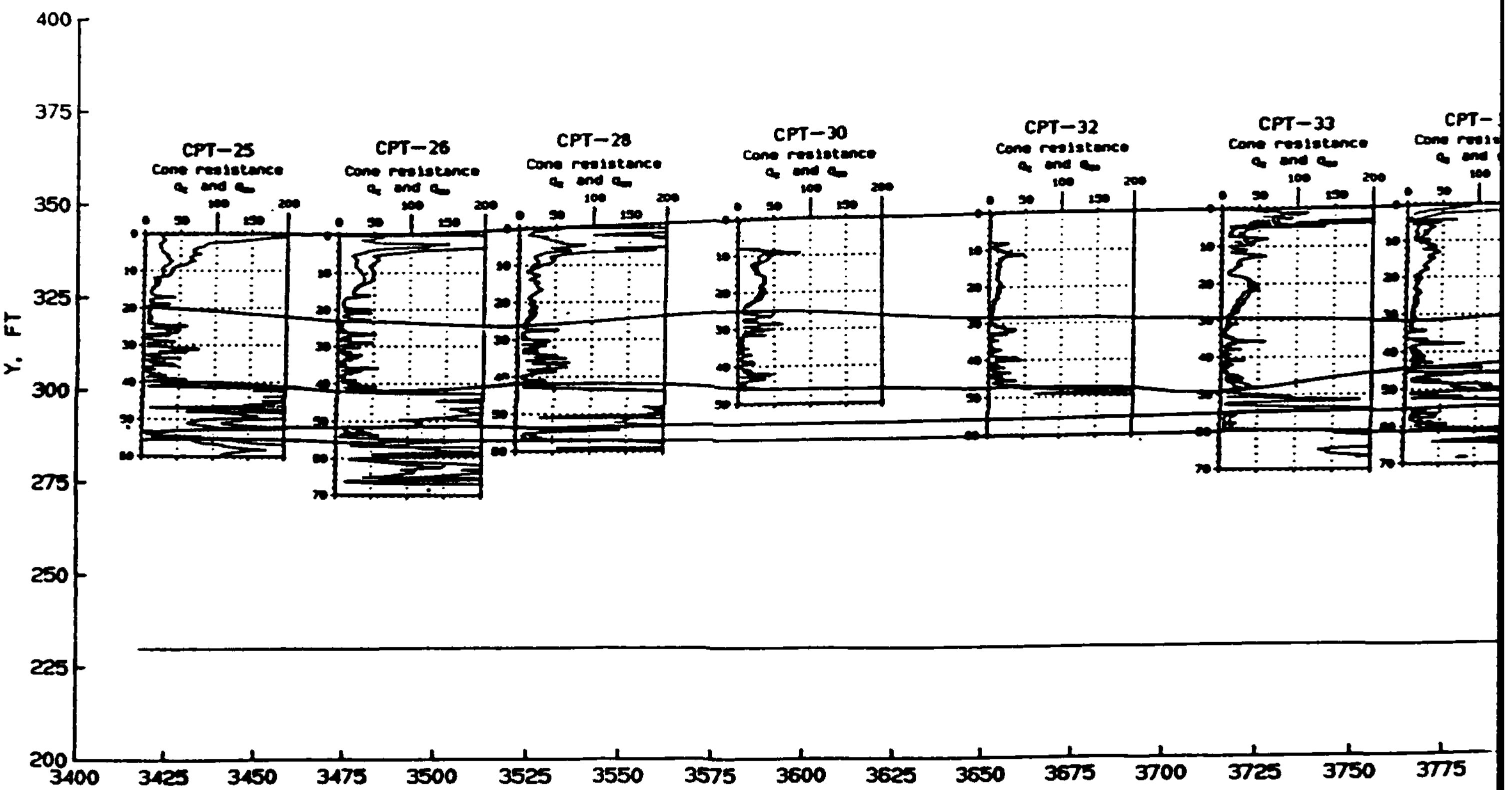
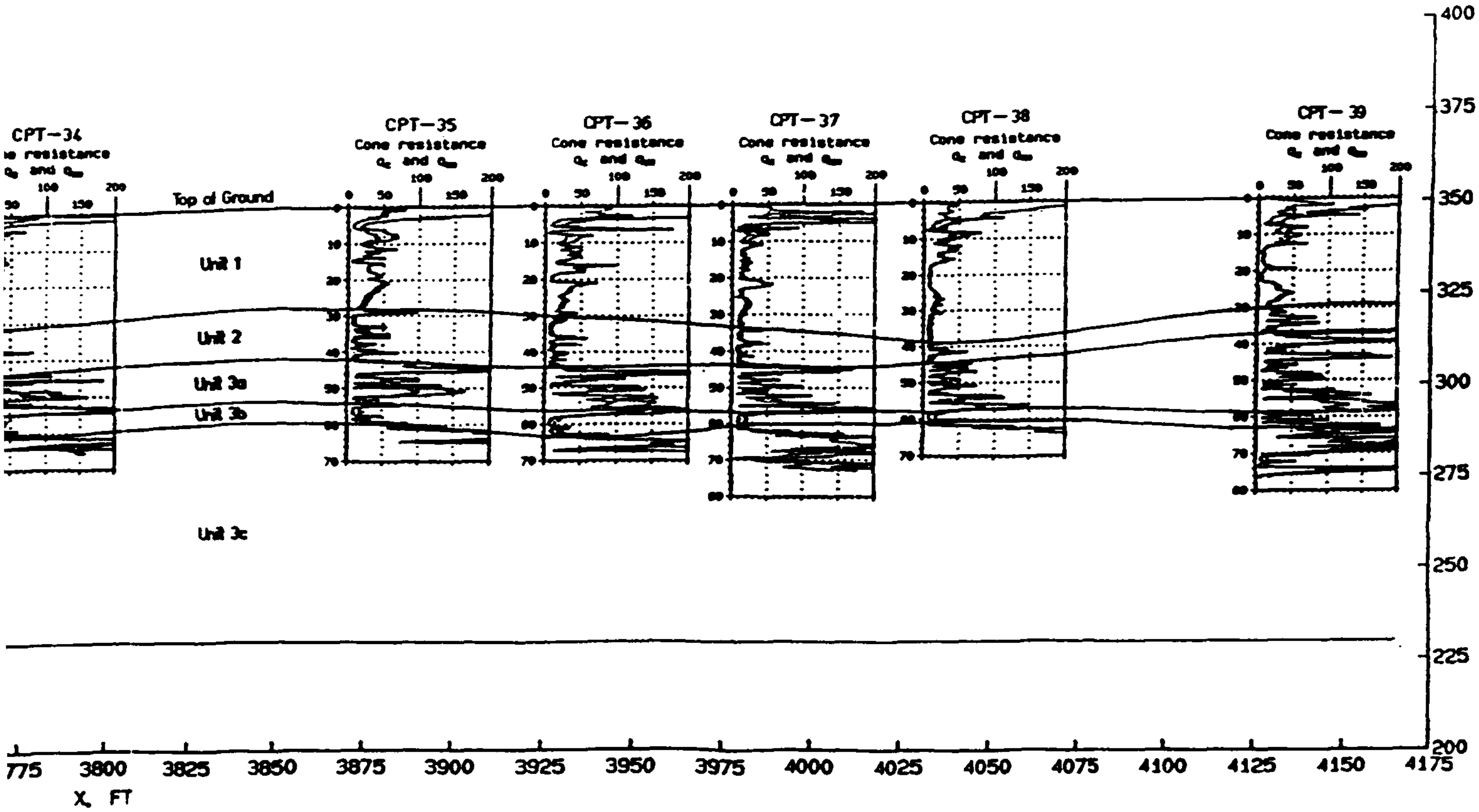


Figure 22. CPT cone resistan
(Running parallel to axis a



.stances along section B'–B'
:is along switchyard berm).

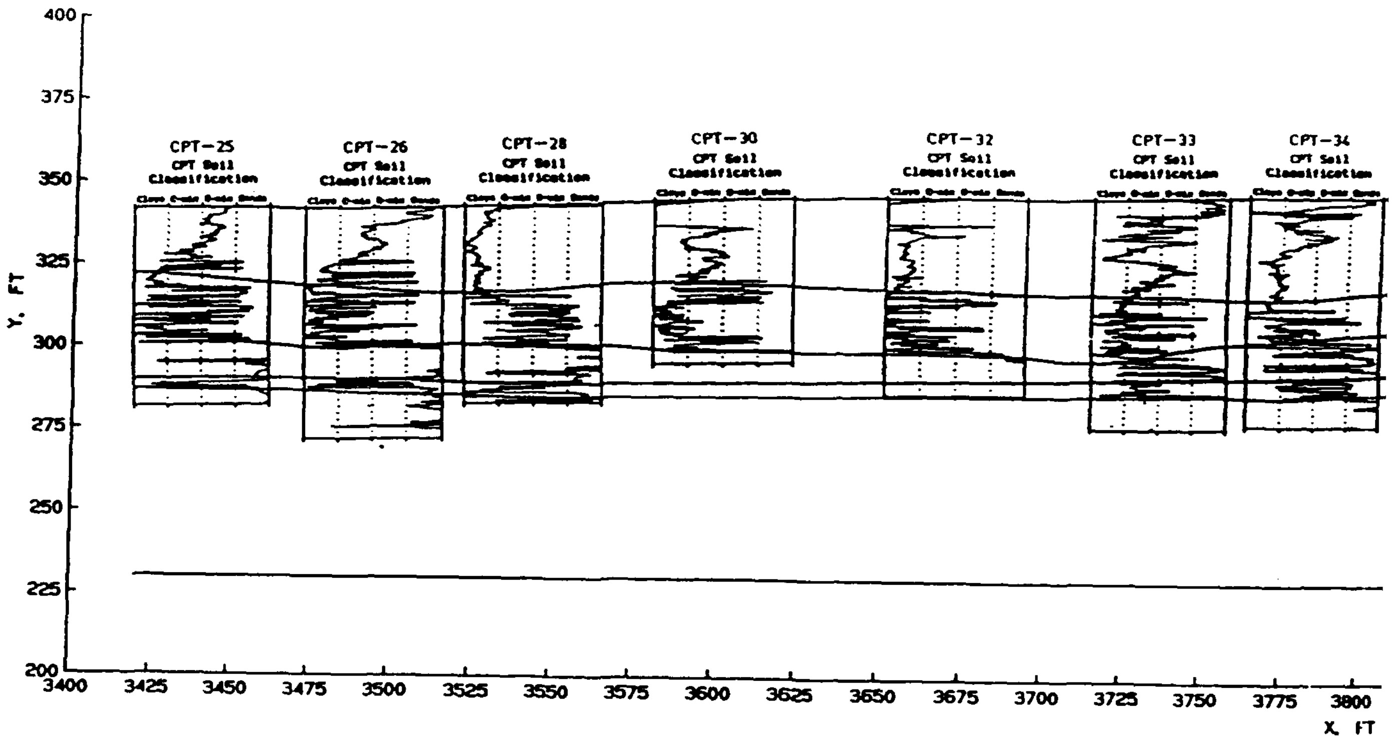
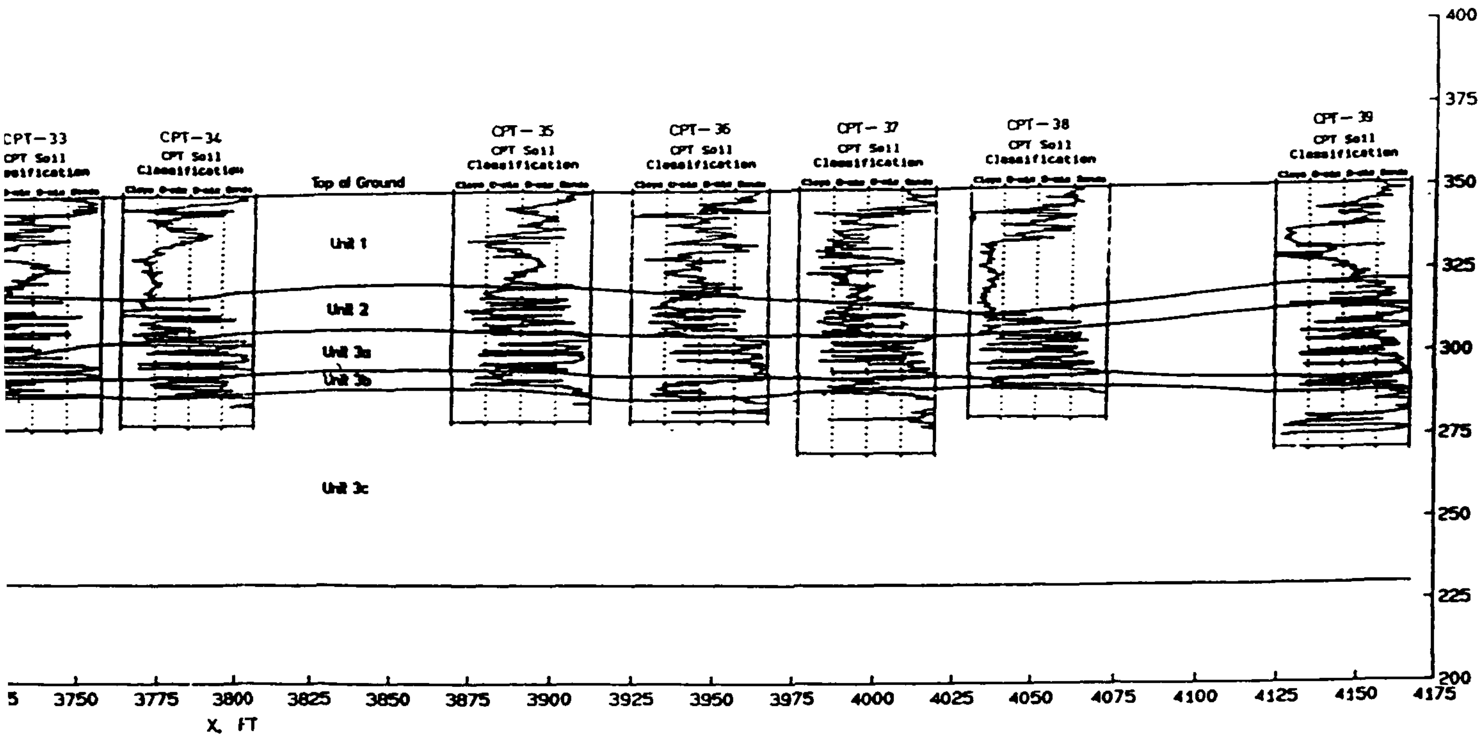


Figure 23. CPT soil classificatio
(Running parallel to axis alor



il classifications along section B' -B'
llel to axis along switchyard berm).

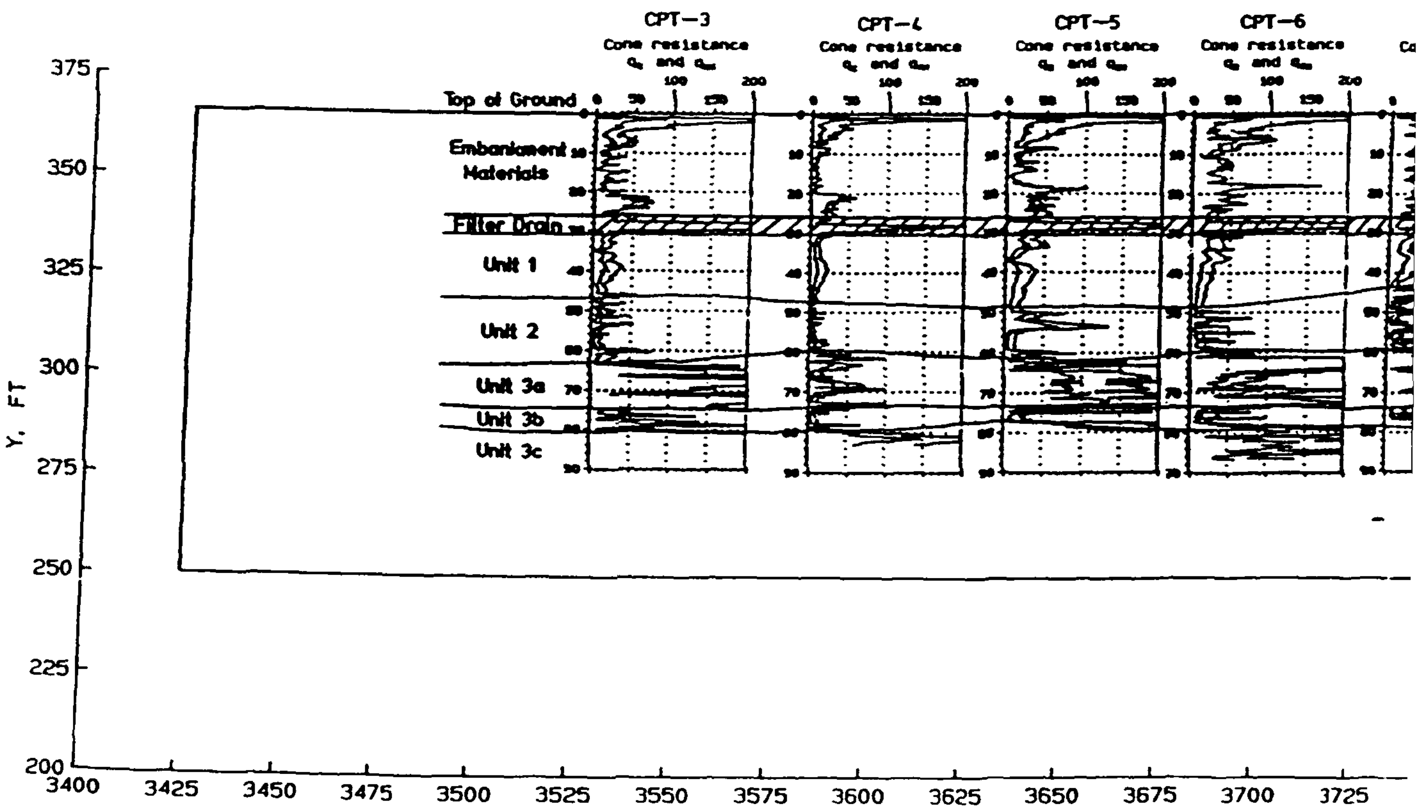
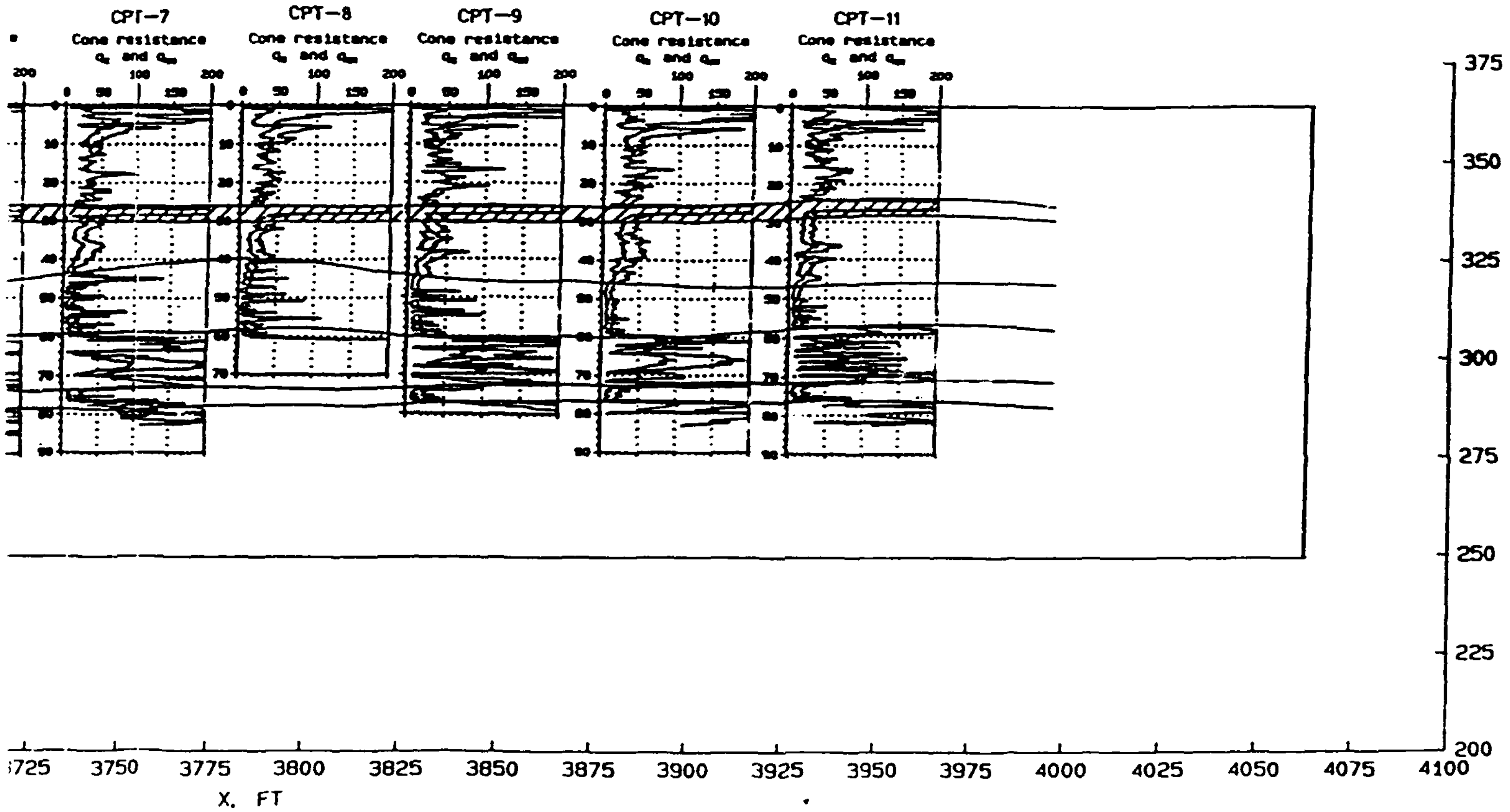


Figure 24. CPT cone res
(Running parallel to



e resistance along section A'-A'
l to axis on switchyard berm).

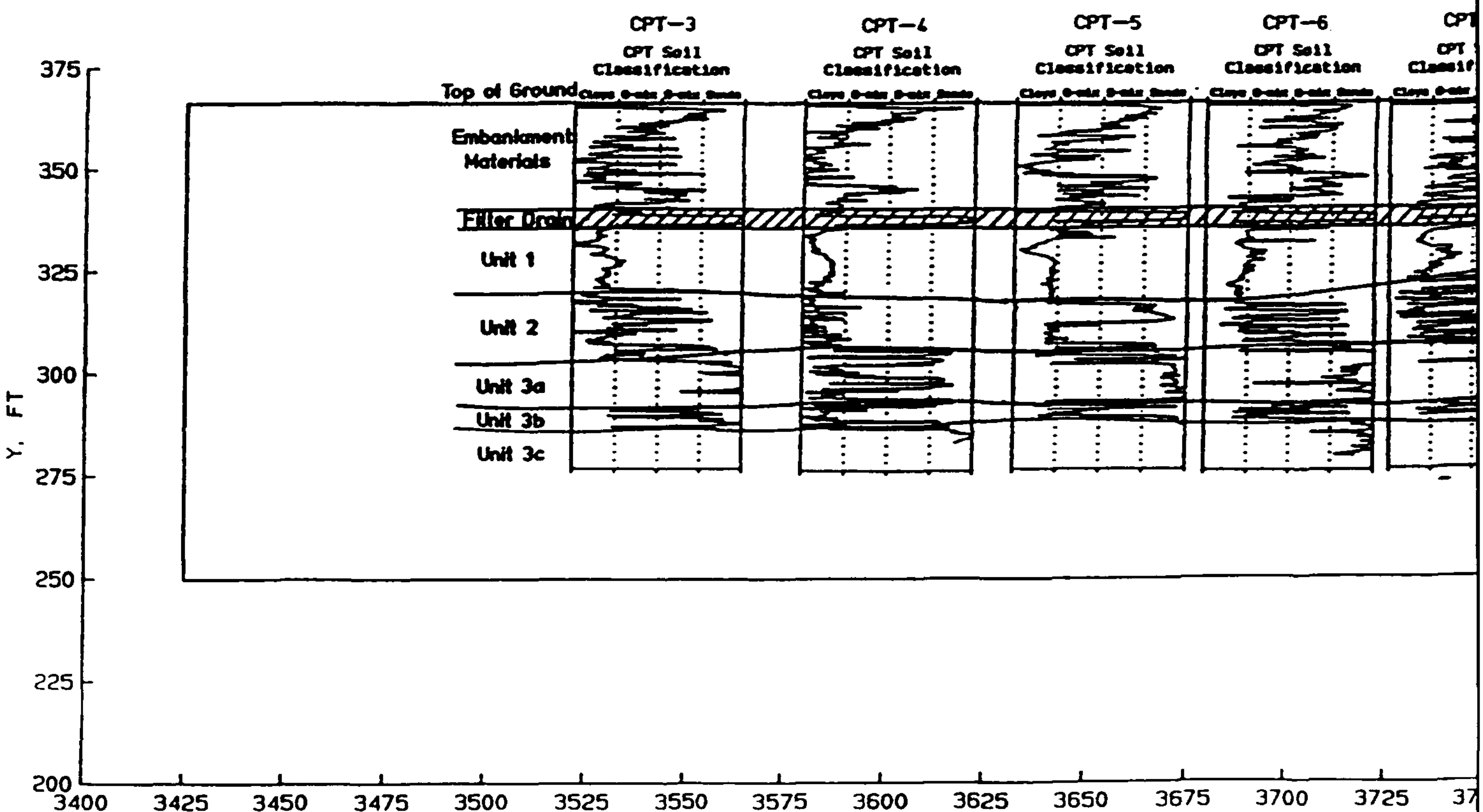
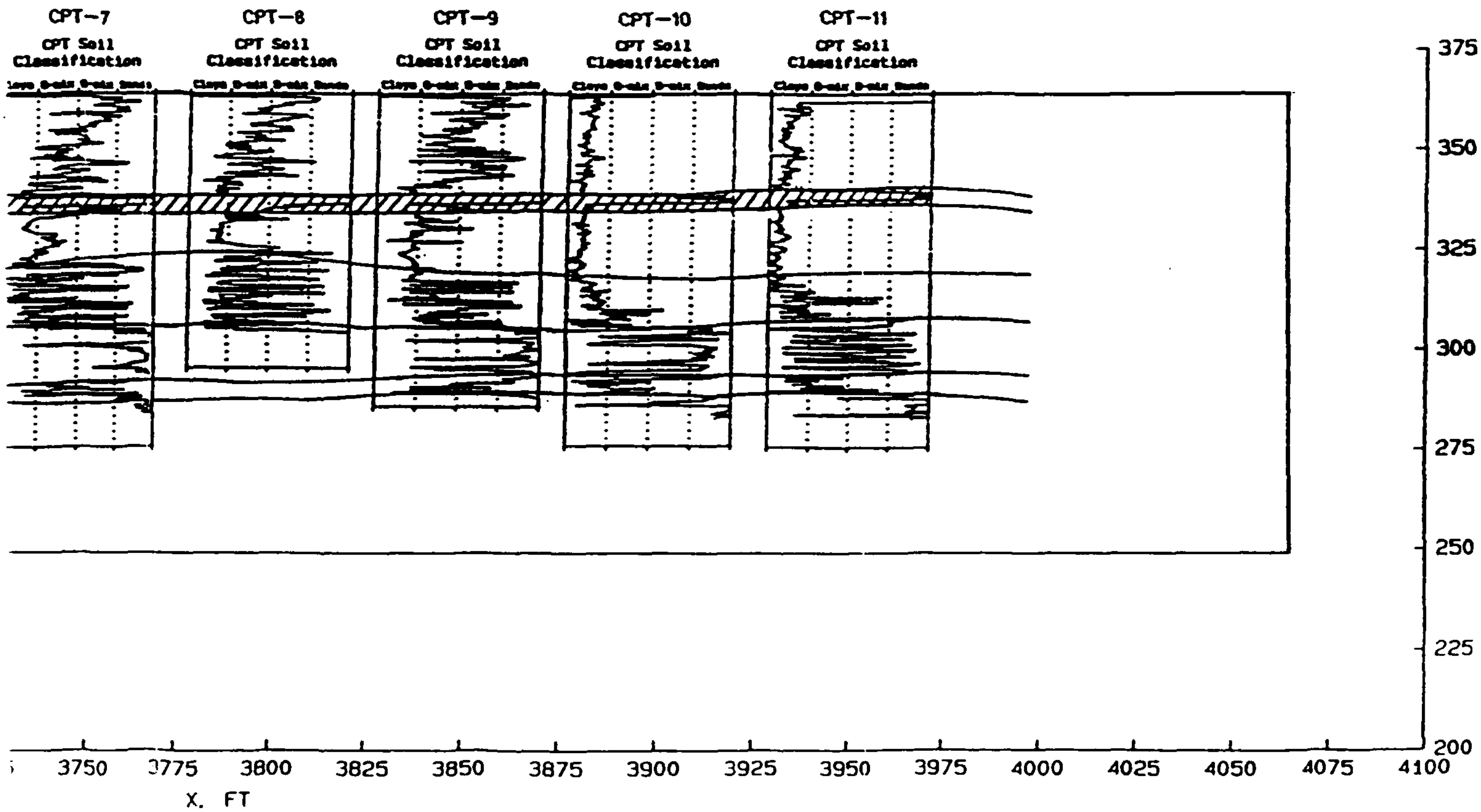


Figure 25. CPT soil classif
(Running parallel to ax



.ssifications along section A'-A'
:o axis on switchyard berm).

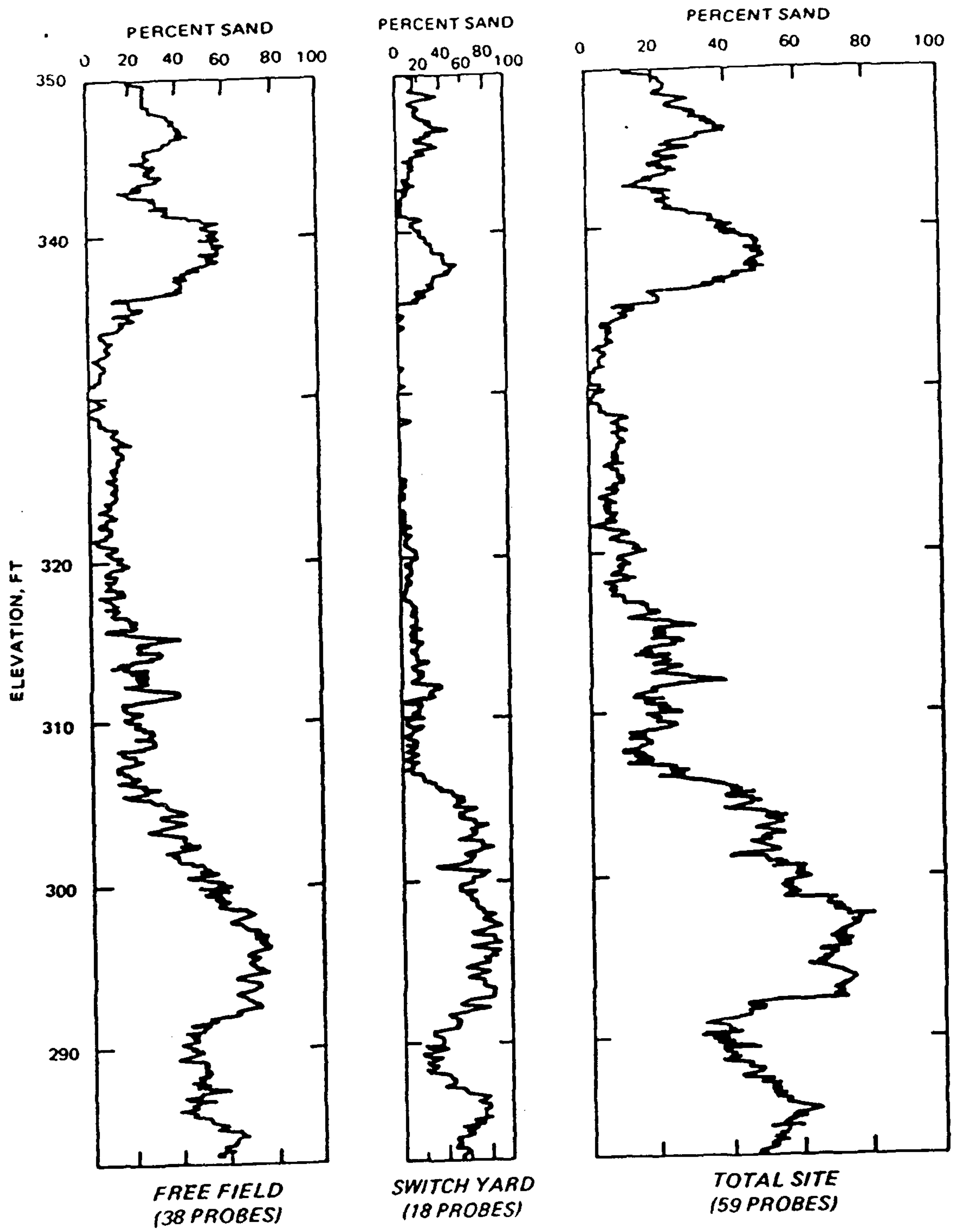


Figure 26. Sand percentages as function of elevation across the site.

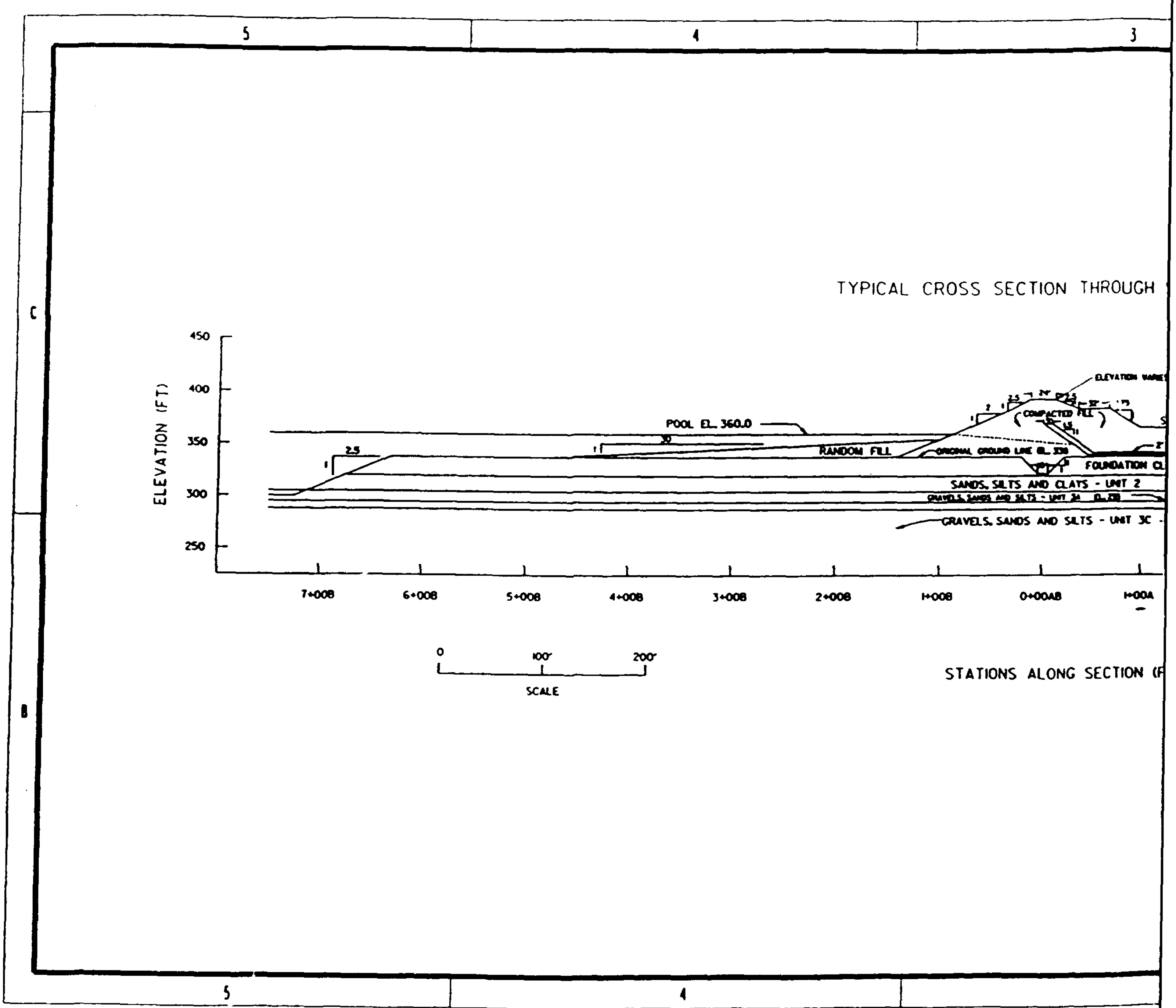
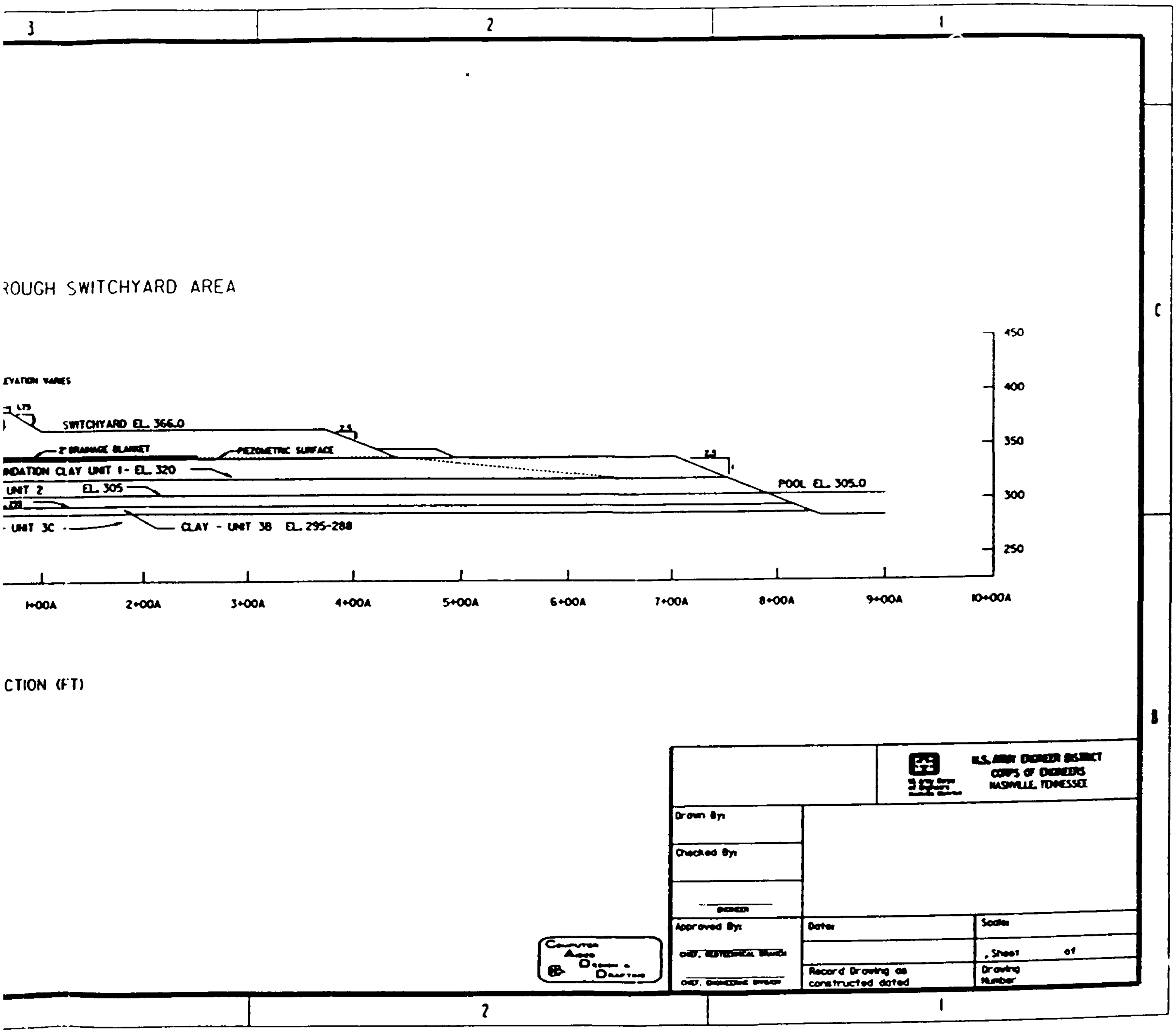


Figure 27. Idealized cross sect



s section of the switchyard area.

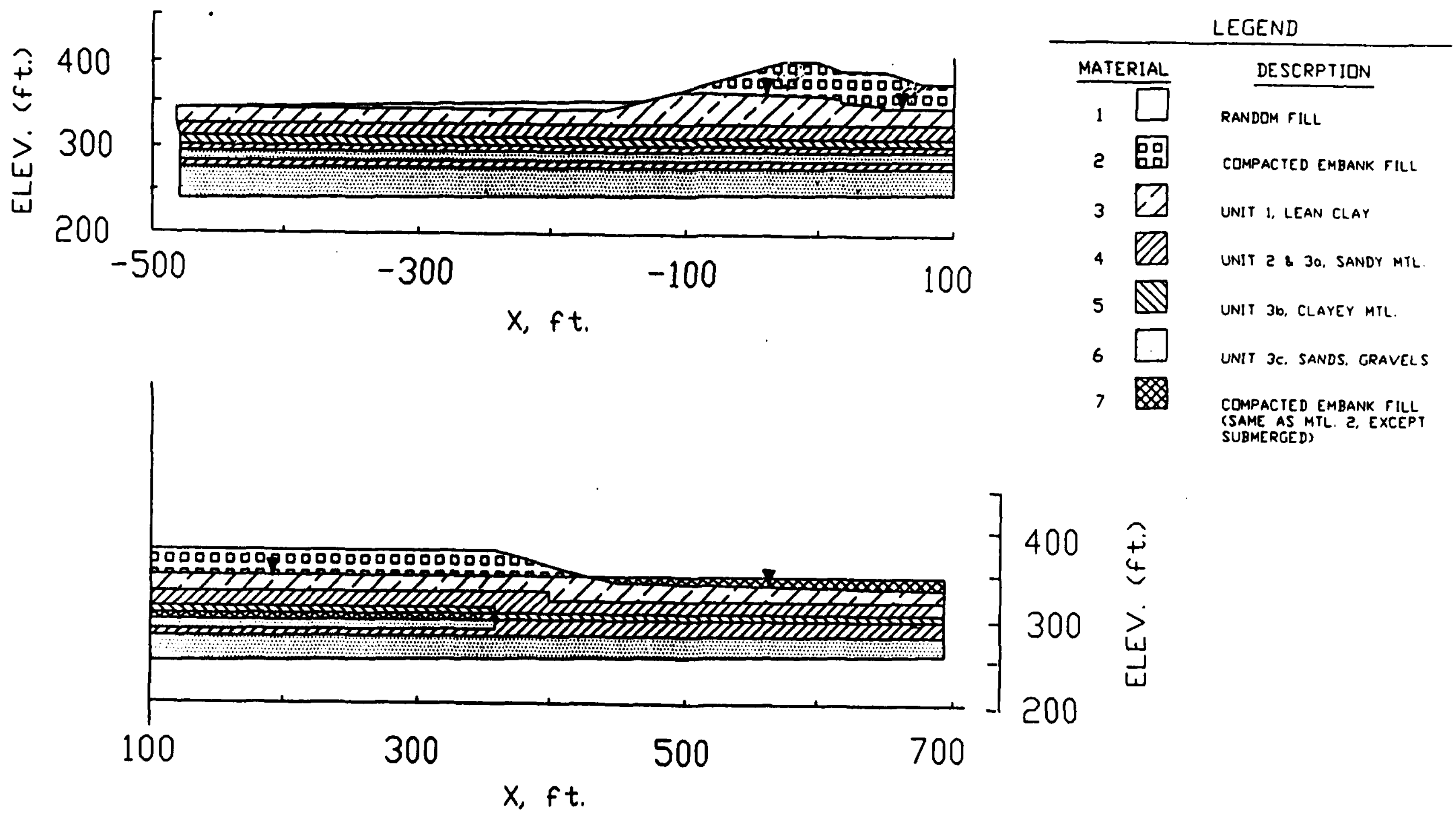


Figure 28. Material distribution for static analysis.

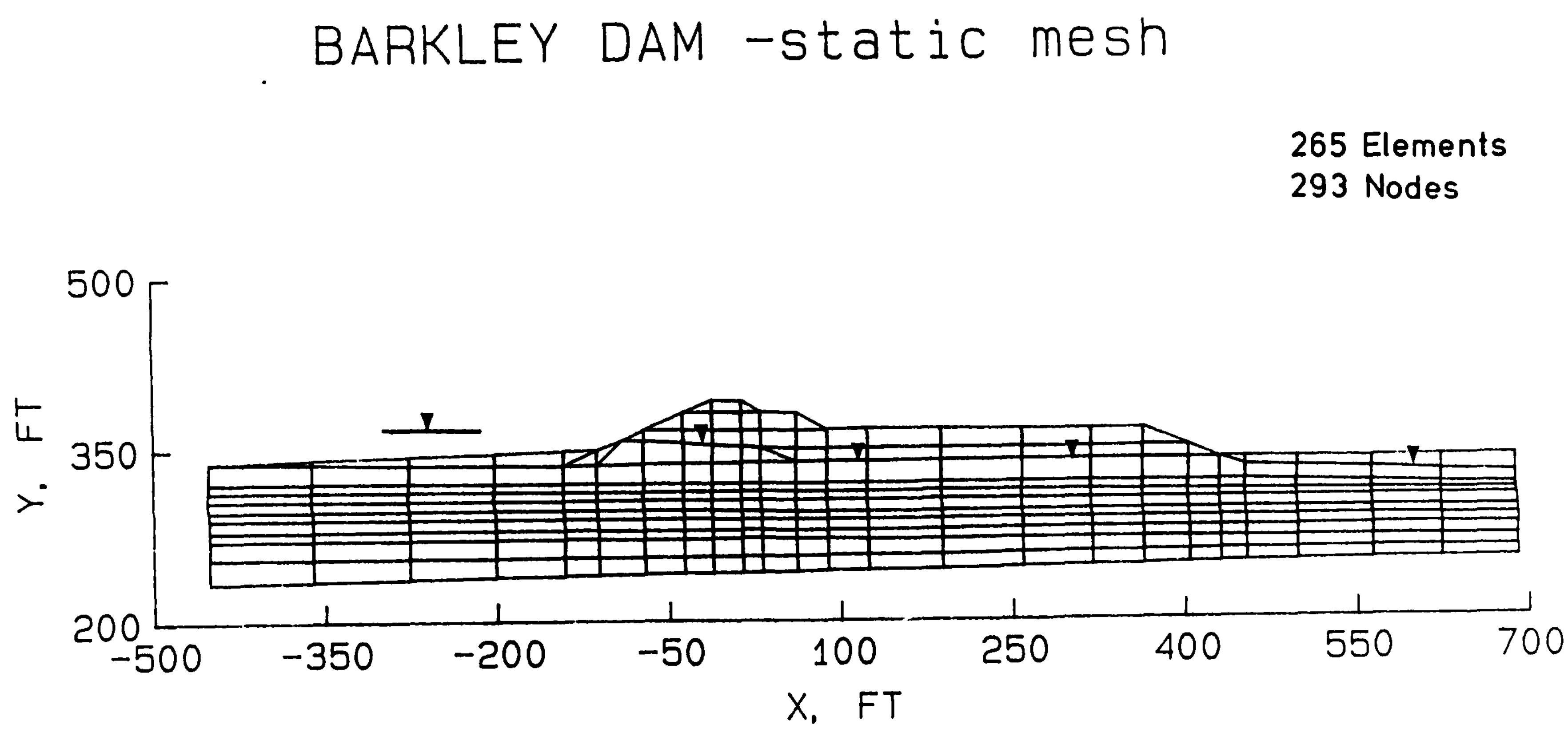


Figure 29. Finite element mesh used in static analysis.

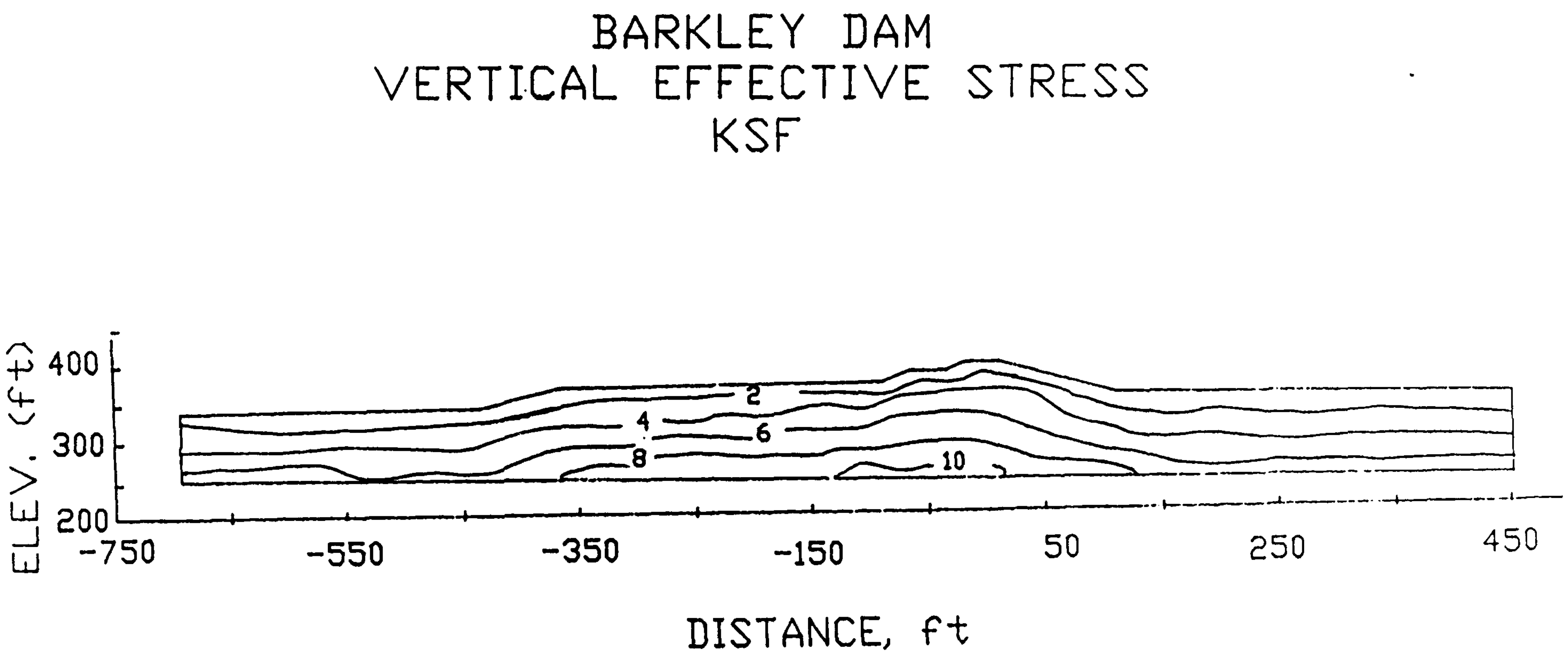


Figure 30. Contours of vertical effective stress, KSF.

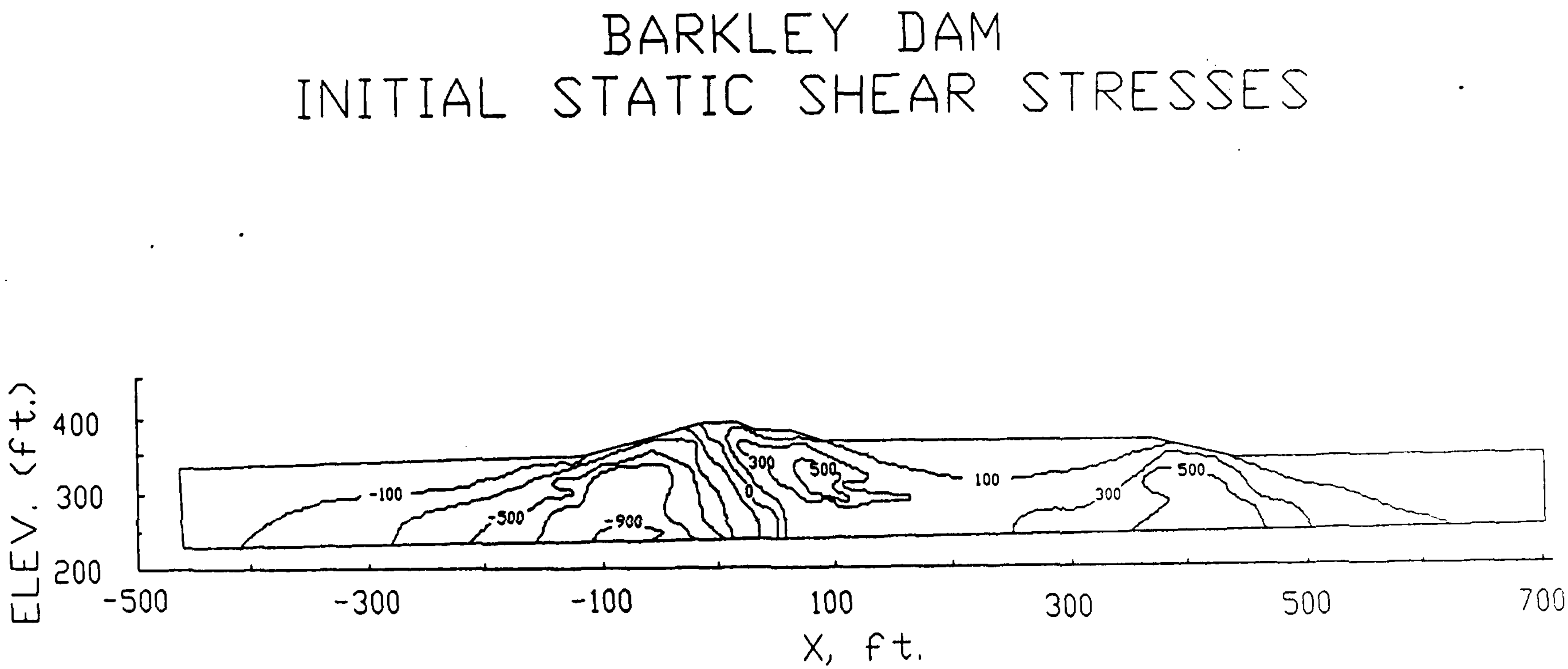


Figure 31. Contours of shear stress on horizontal planes, KSF.

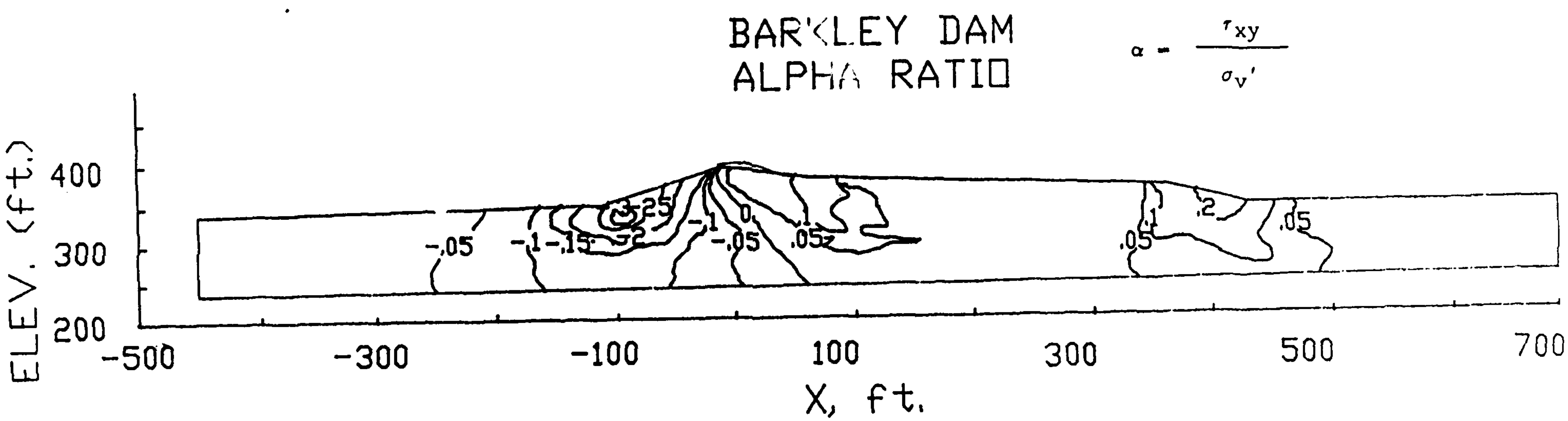


Figure 32. Alpha Ratio.

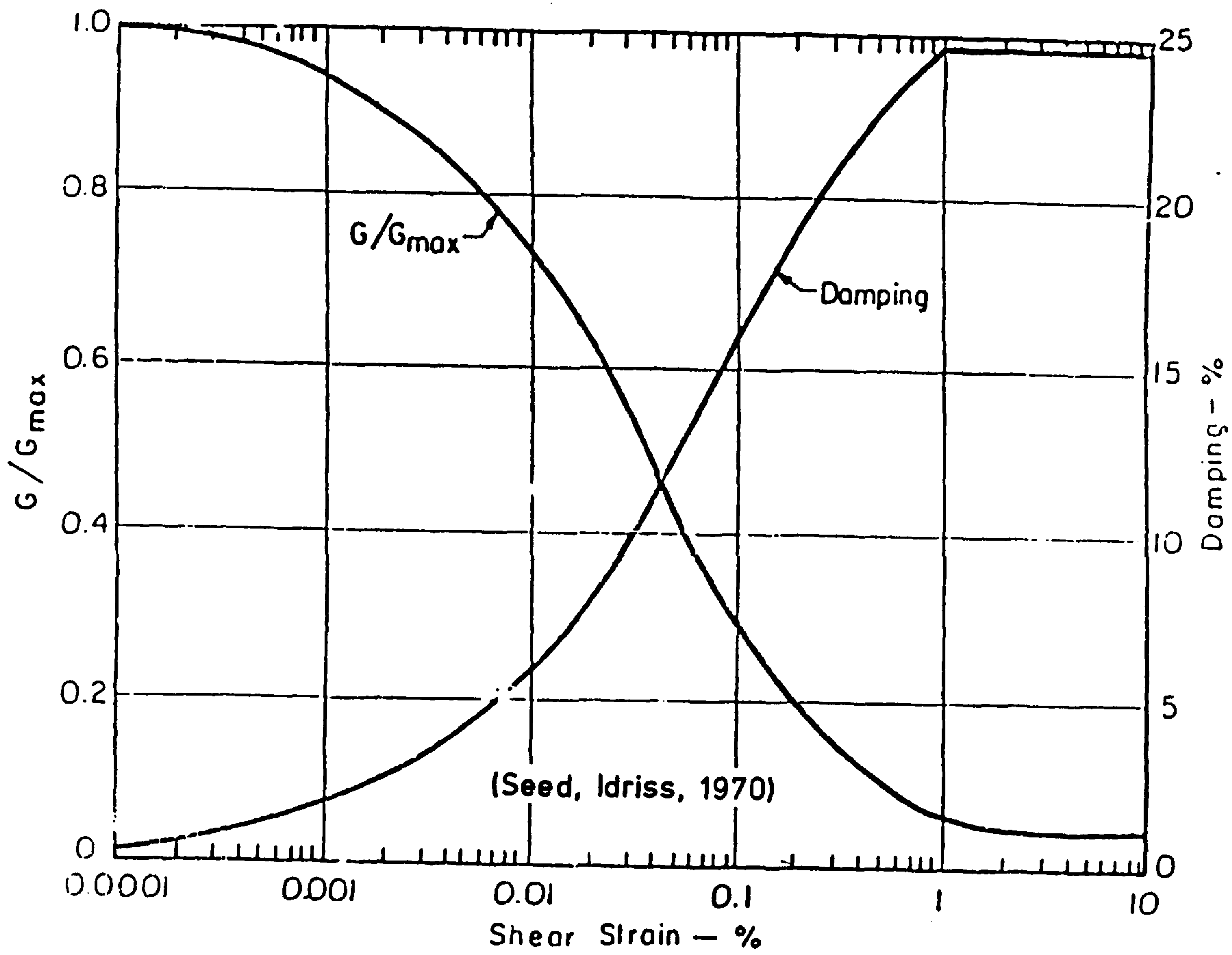


Figure 33. Strain dependent modulus degradation and damping curves used by SHAKE and FLUSH.

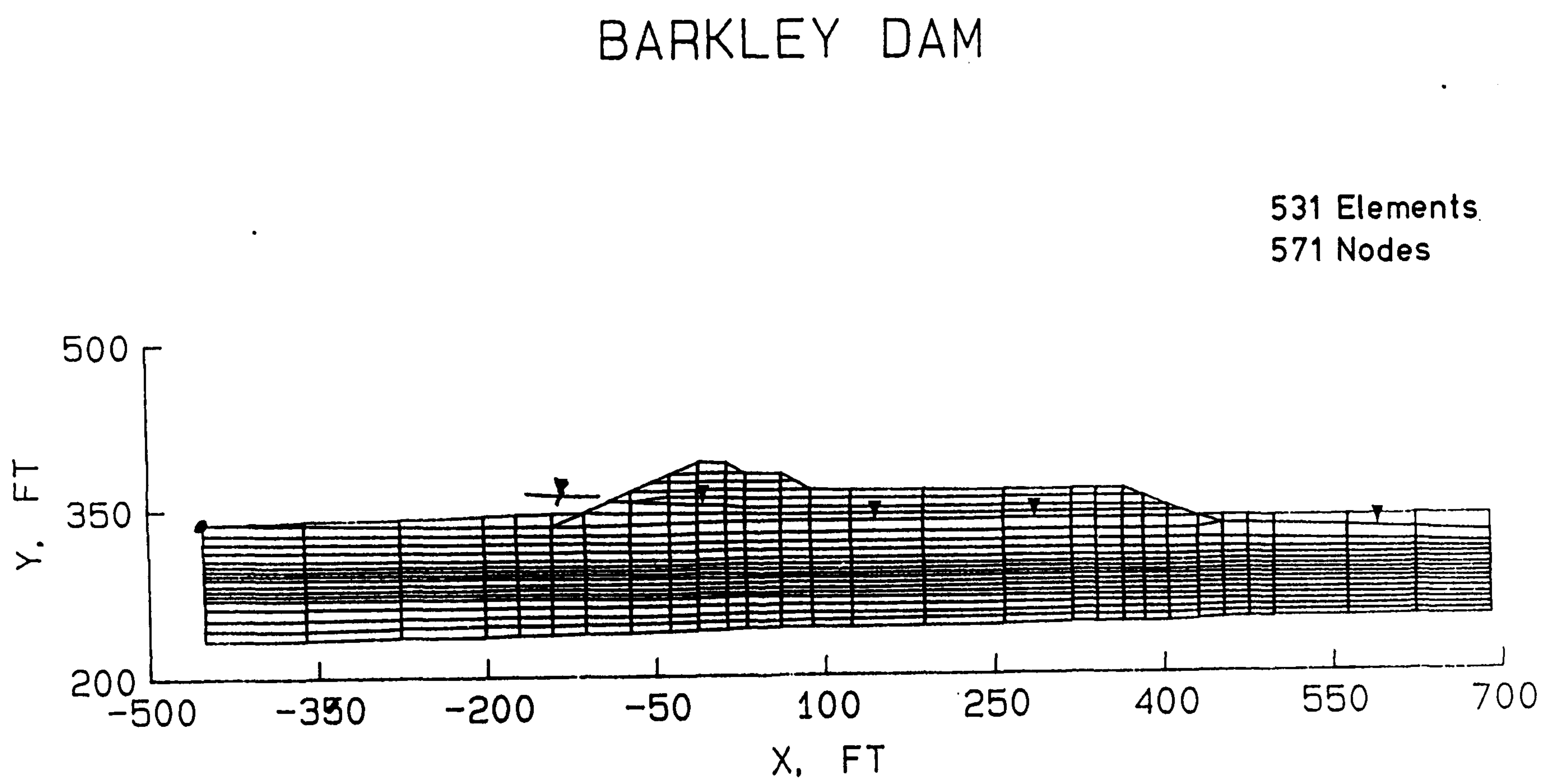


Figure 34. Finite element mesh for dynamic analysis.

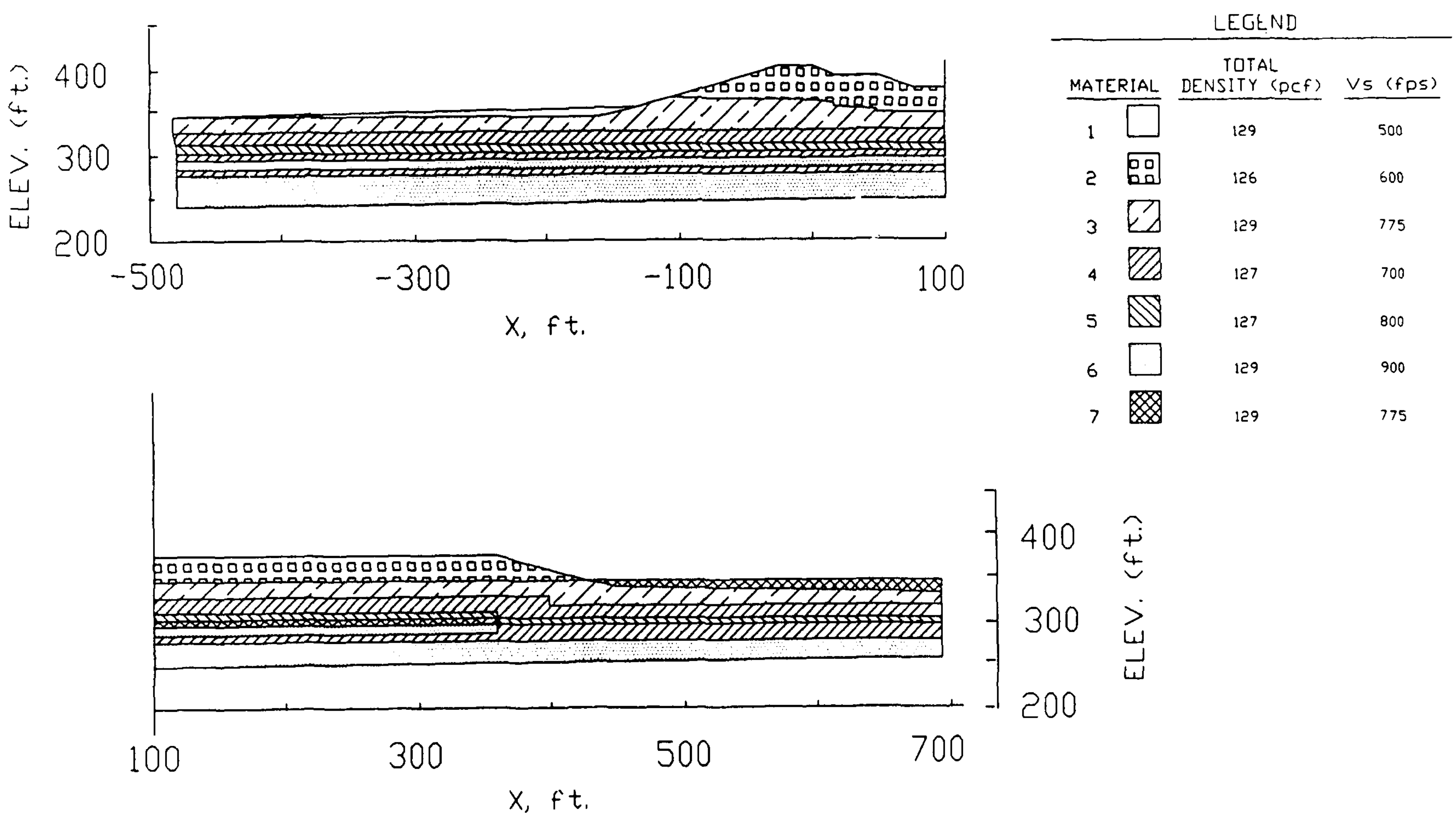


Figure 35. Dynamic material properties.

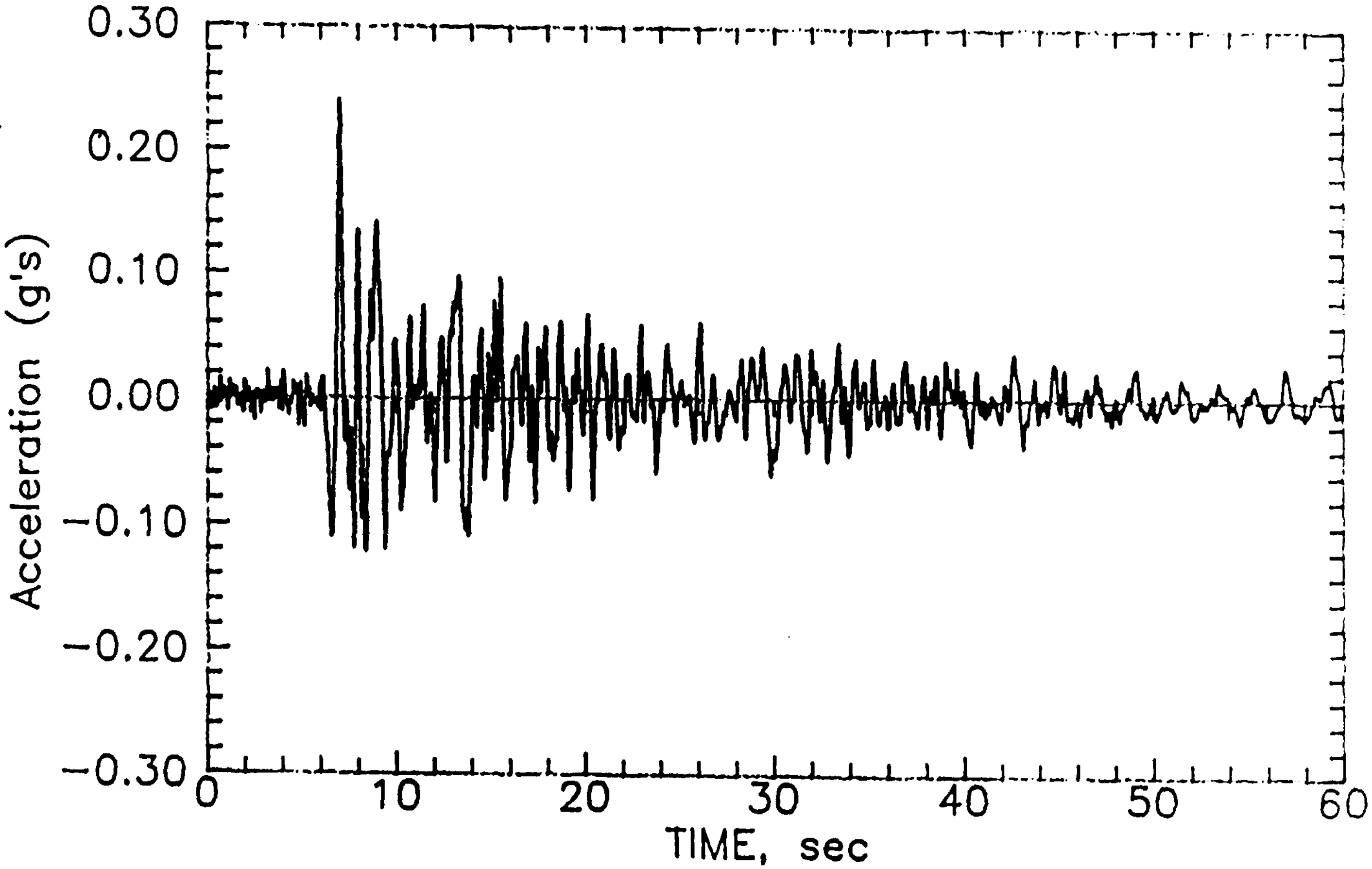


Figure 36. Design accelerogram - S48°E component of the Santa Barbara Courthouse record of the Kern County Earthquake of July 21, 1952.

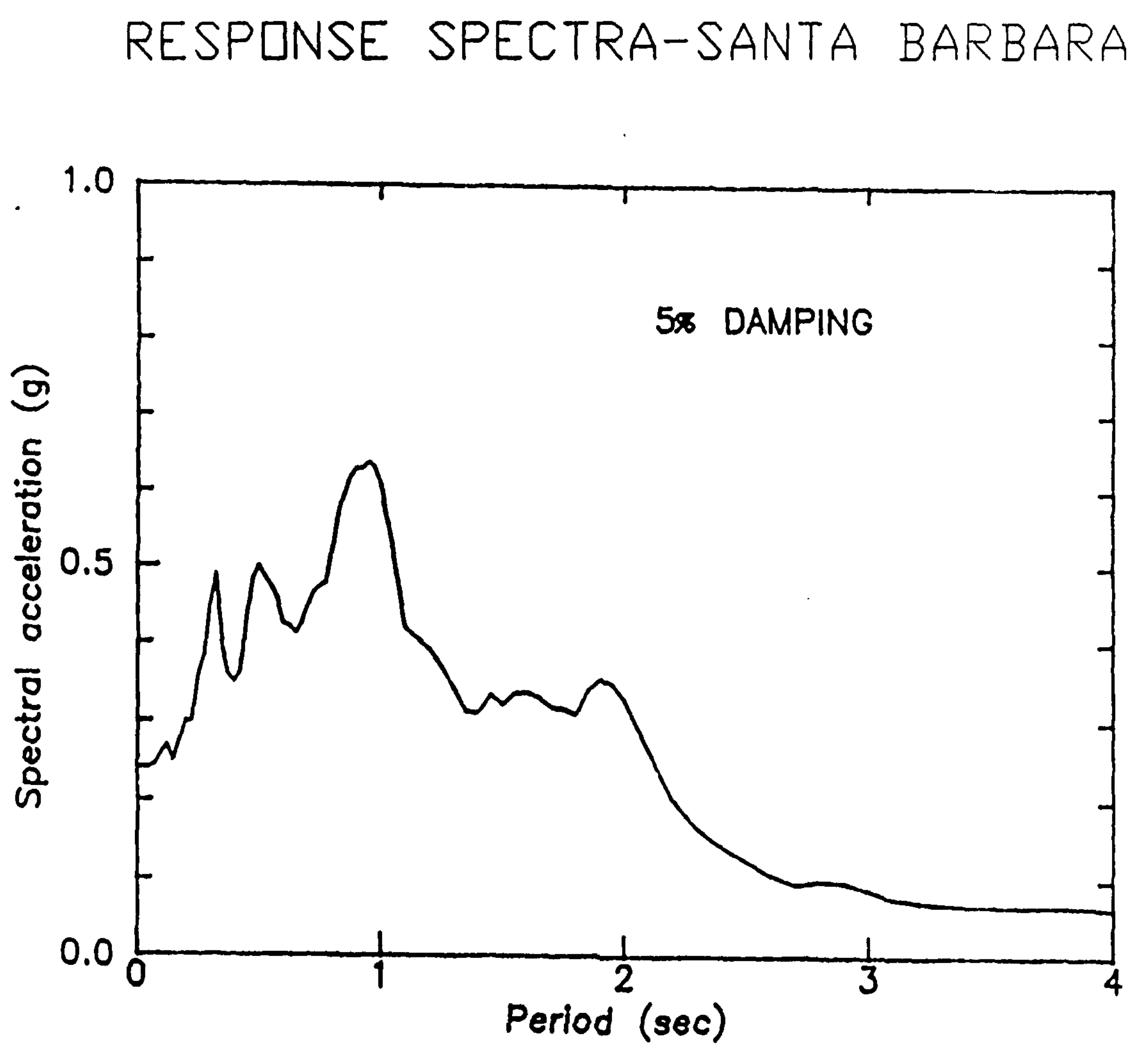


Figure 37. Acceleration response spectra of the S48°E component of the Santa Barbara Courthouse record of July 21, 1952.

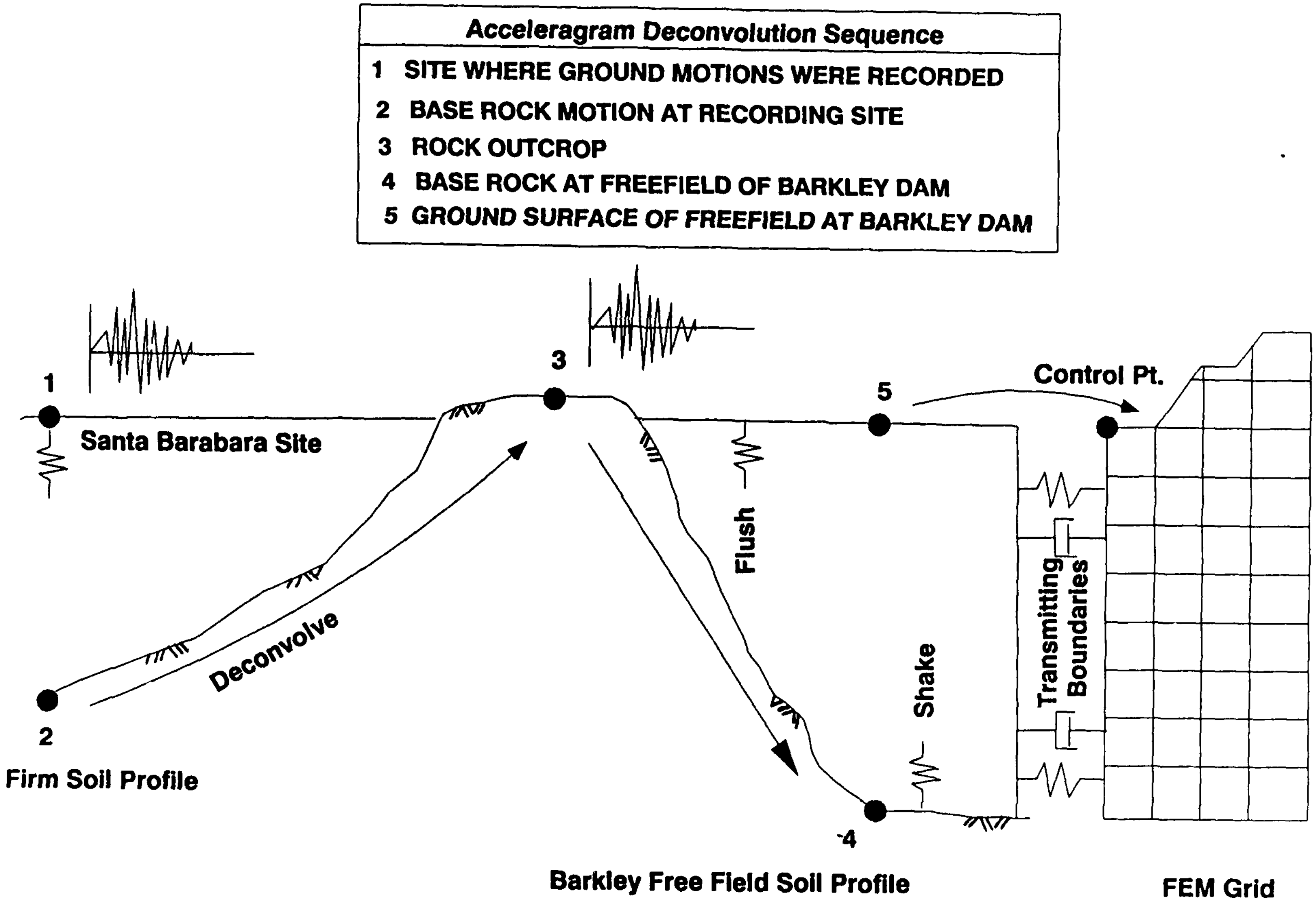


Figure 38. Schematic showing how Input Acceleration History was derived.

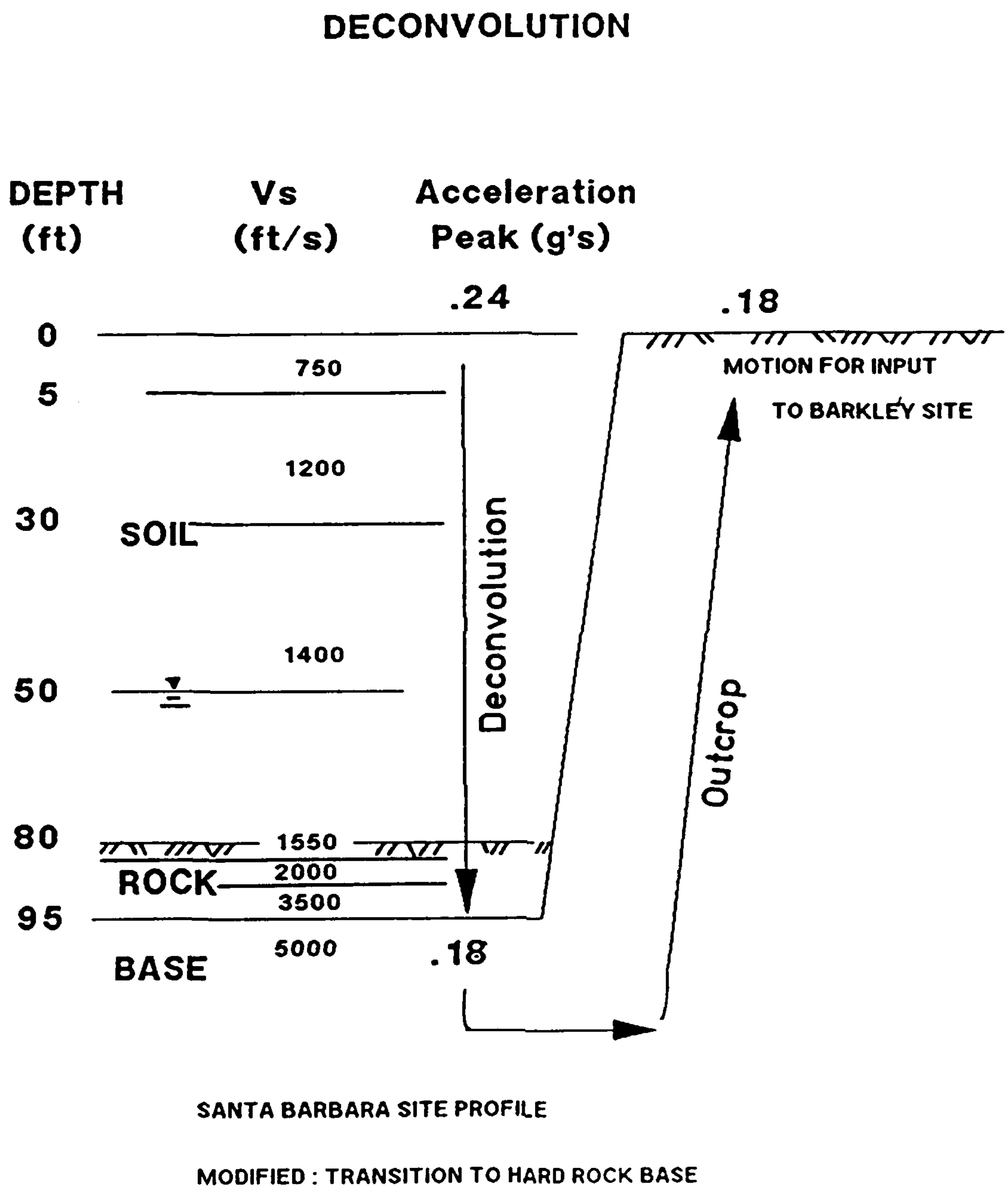


Figure 39. Santa Barbara soil profile.

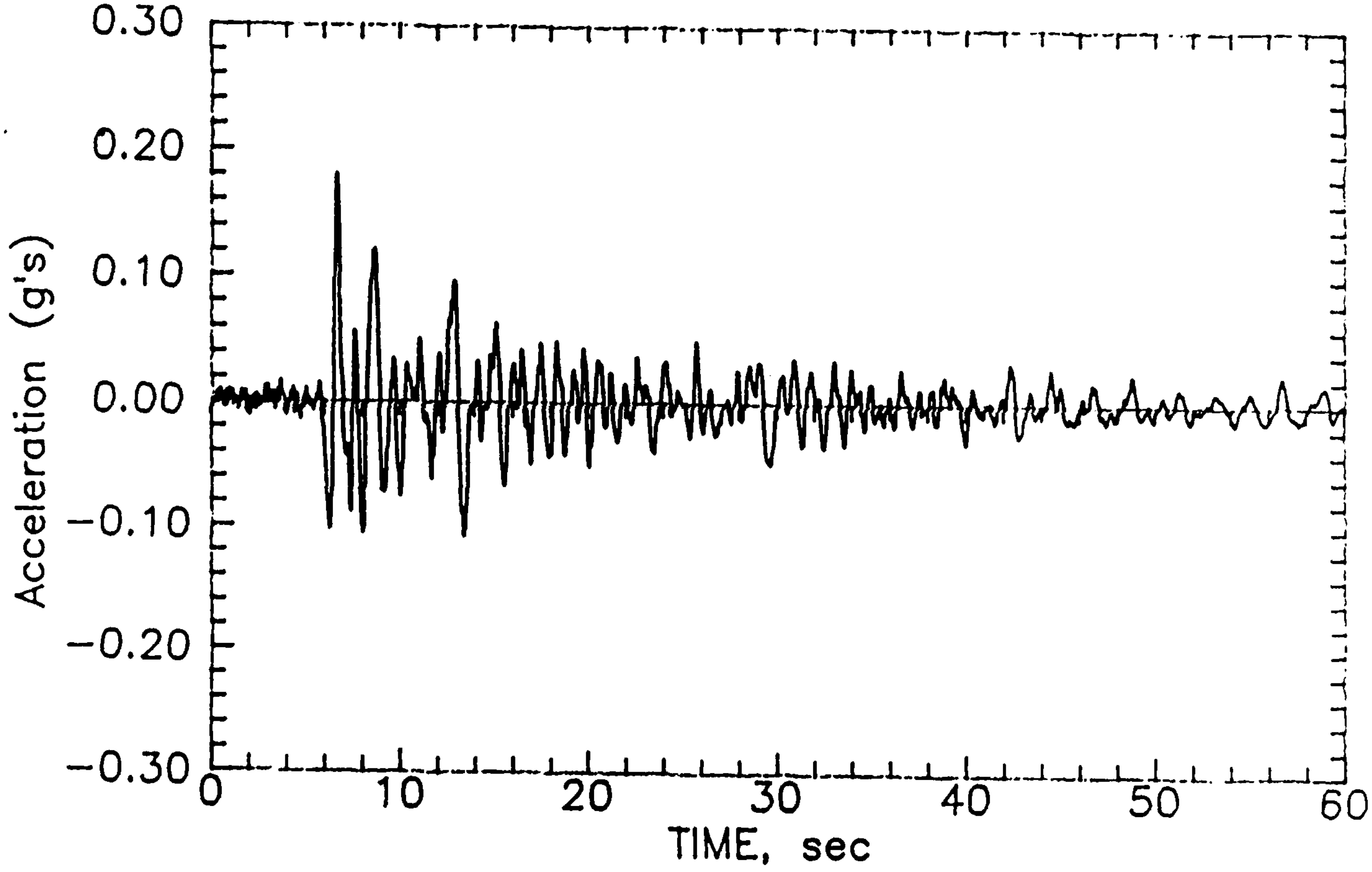


Figure 40. Acceleration history at Point #3.

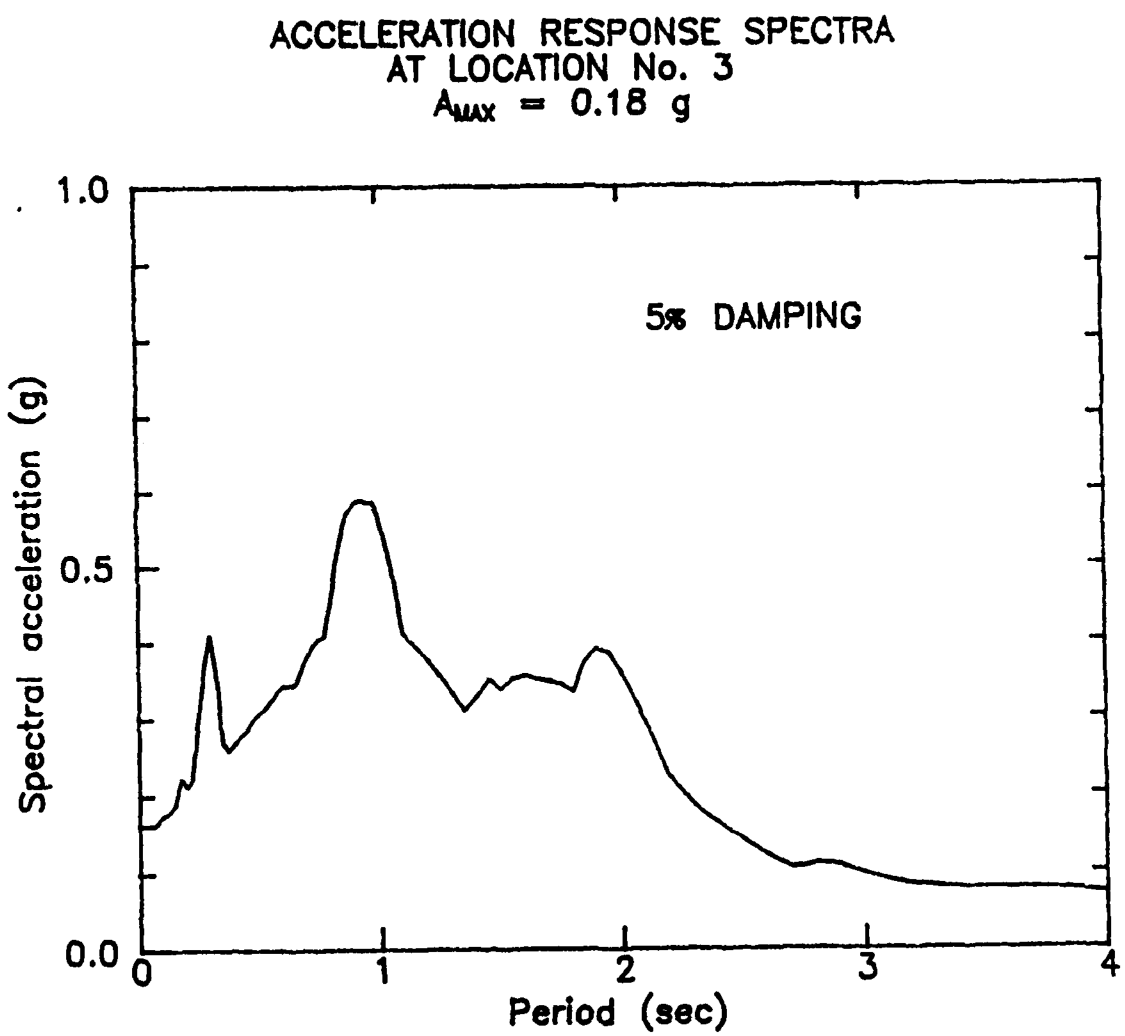


Figure 41. Acceleration response spectra of ground motion at Point #3.

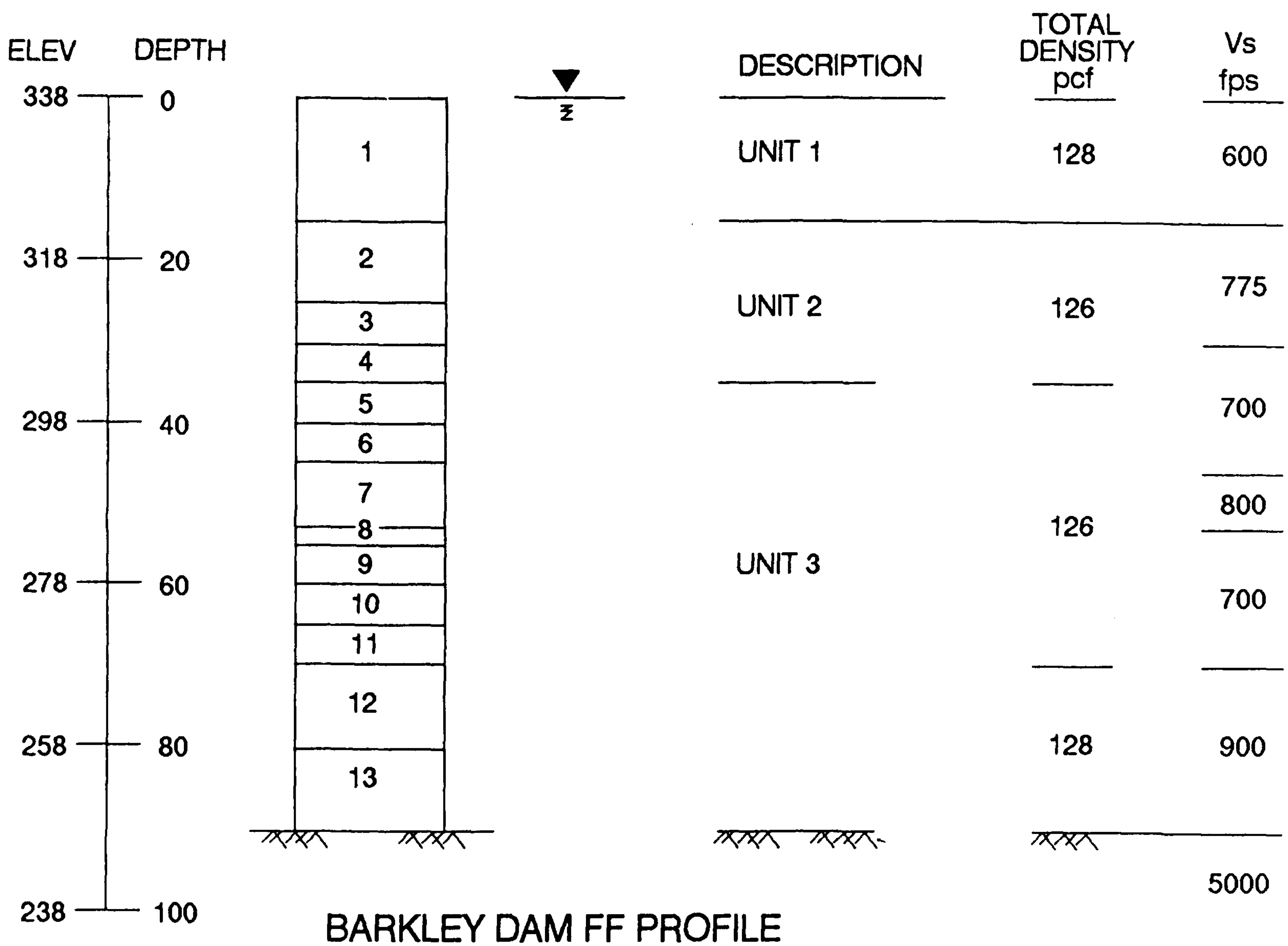


Figure 42. Free field profile at Barkley Dam.

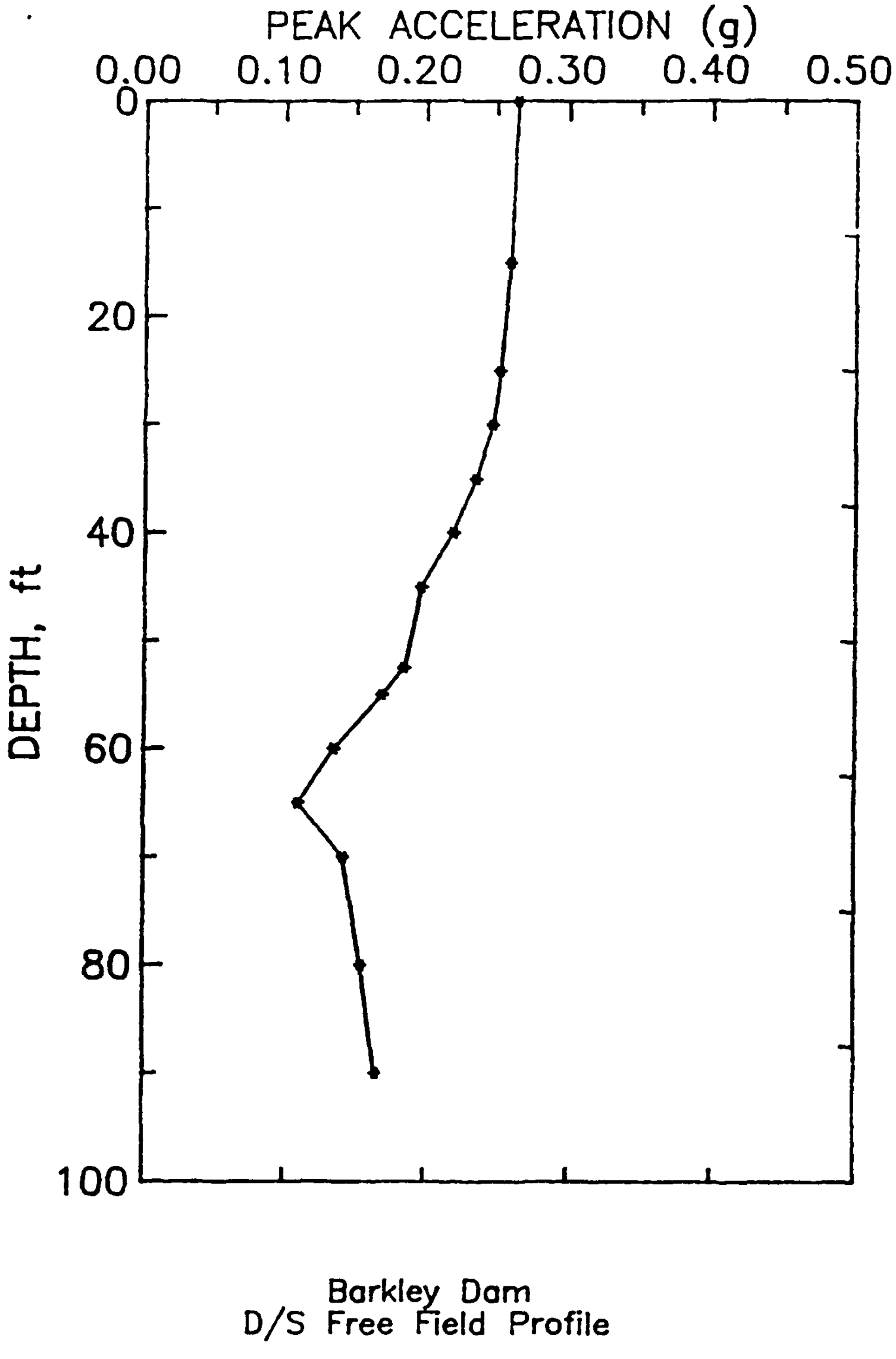


Figure 43. Peak acceleration versus depth for FF profile at Barkley Dam.

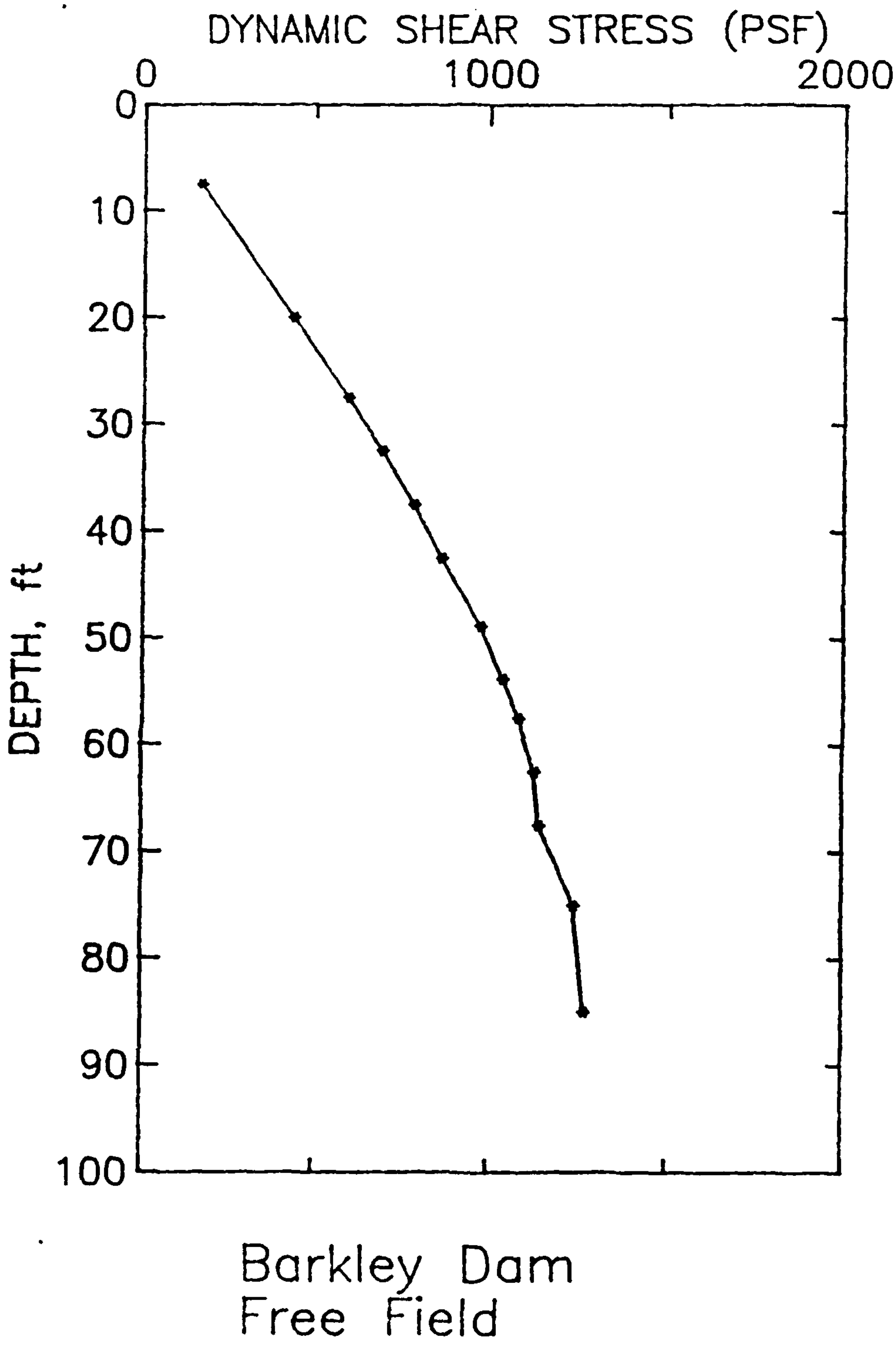


Figure 44. Effective dynamic shear stresses in free field profile at Barkley dam (65% of Maximum).

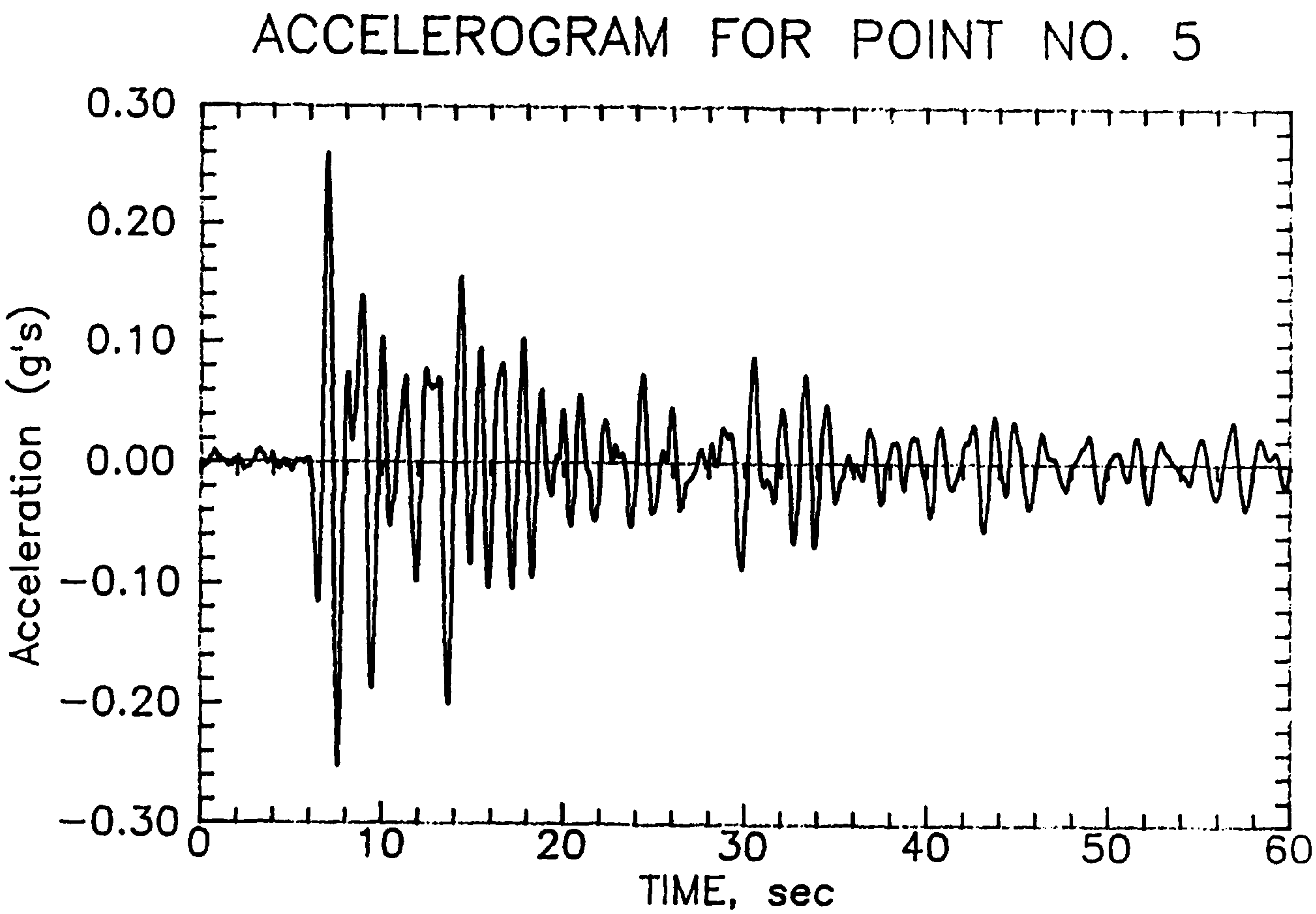


Figure 45. Acceleration for point #5 and input accelerogram for finite element mesh.

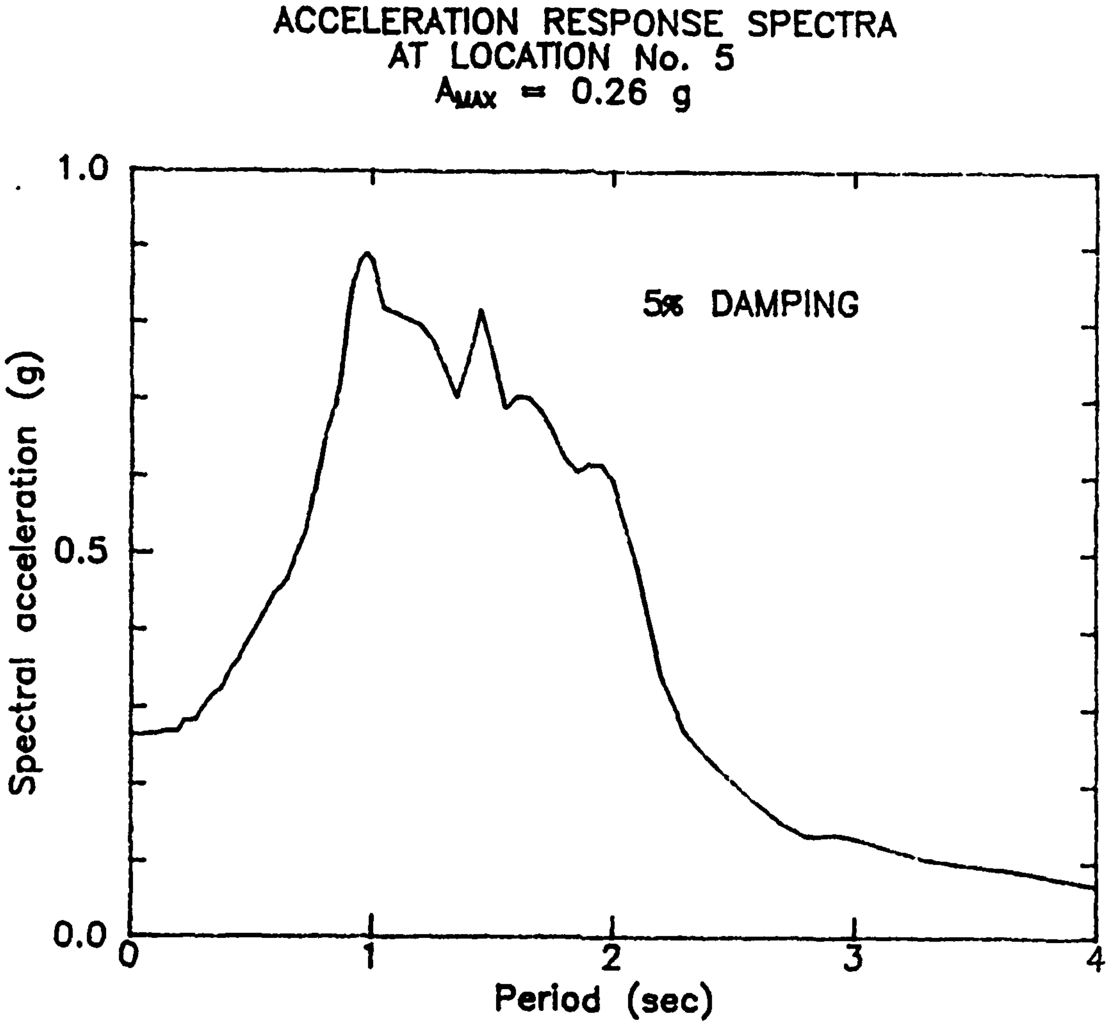


Figure 46. Acceleration response spectra of motions at point #5.

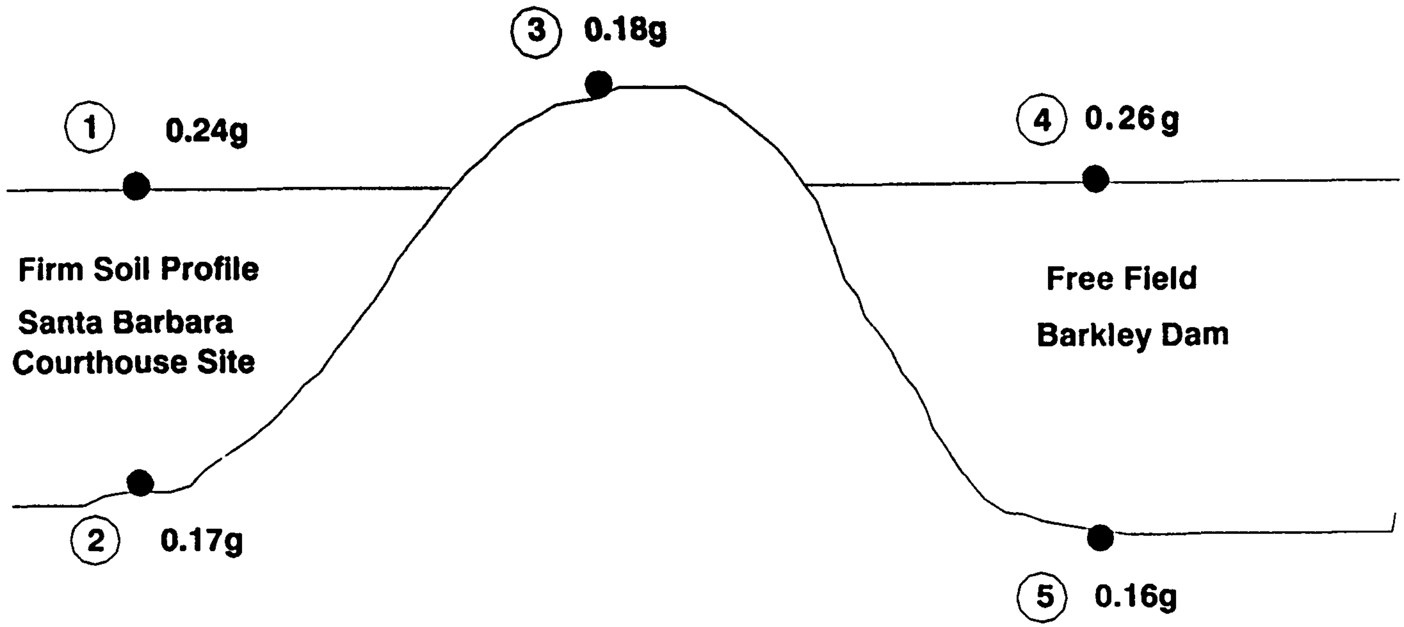


Figure 47. Peak accelerations at key locations.

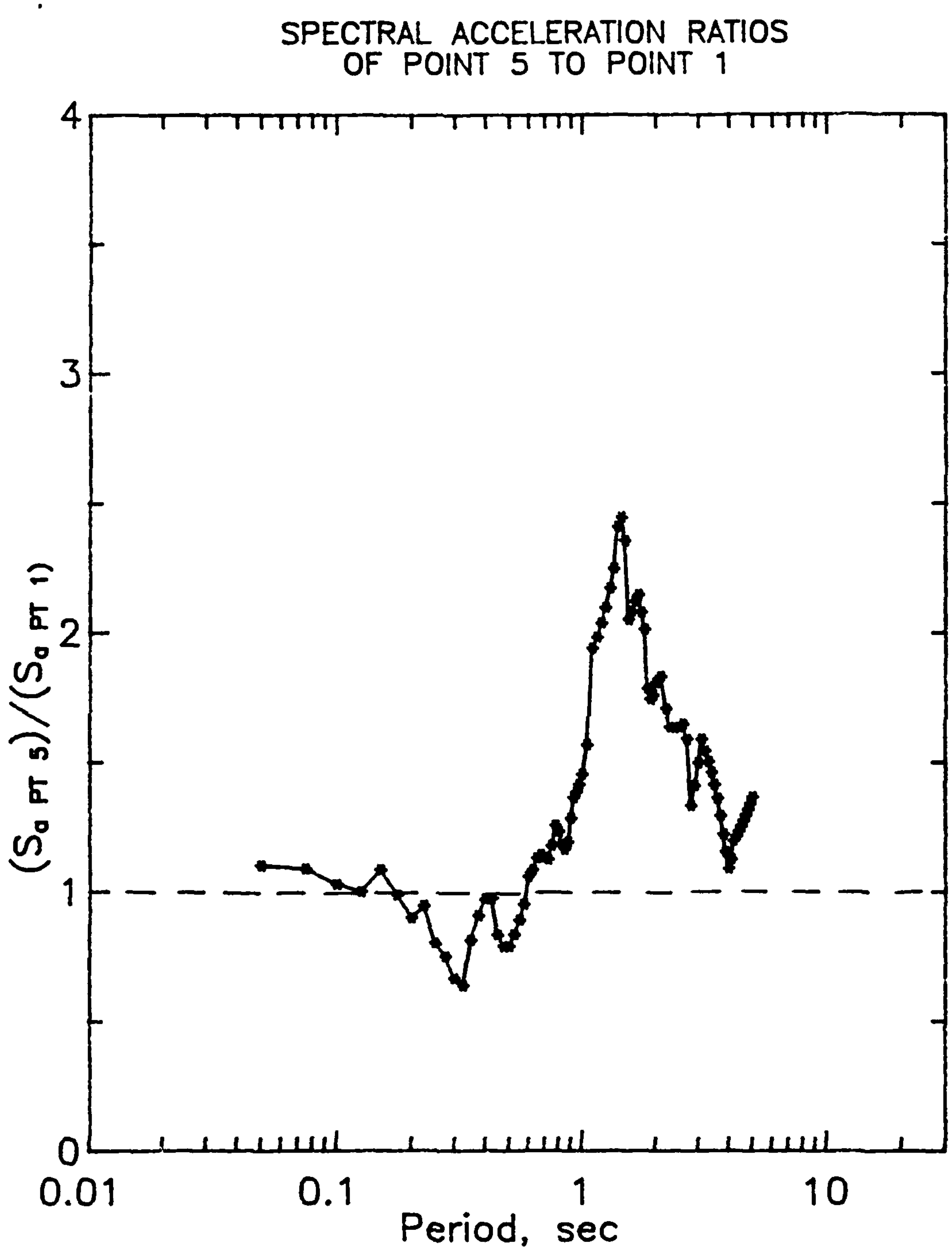


Figure 48. Spectral acceleration ratios of motions at Point #5 to those at Point #1.

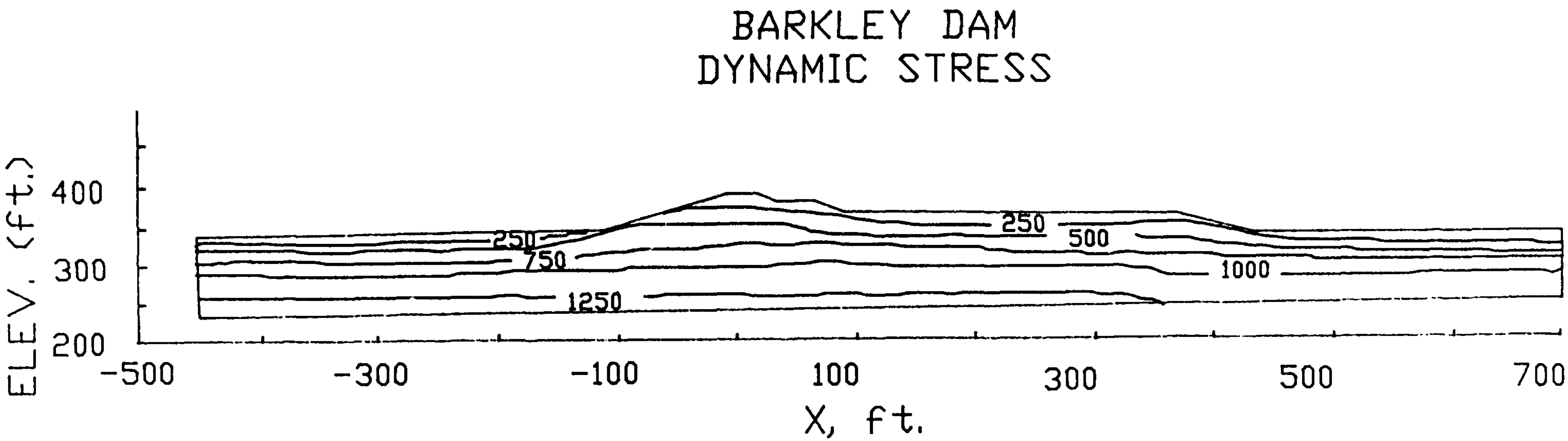


Figure 49. Effective earthquake-induced dynamic shear stresses computed with FLUSH (65% of maximum).

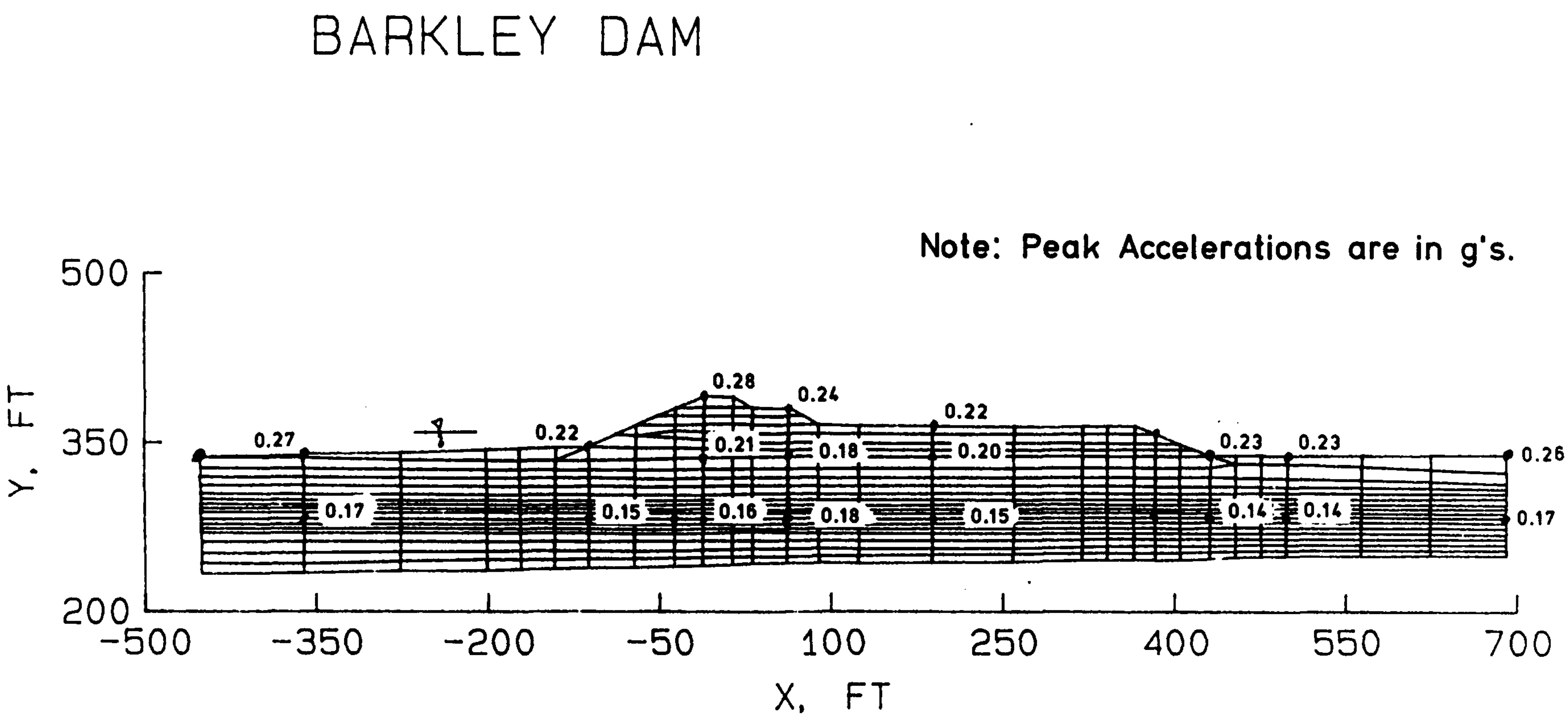


Figure 50. Peak accelerations at selected nodes computed with FLUSH.

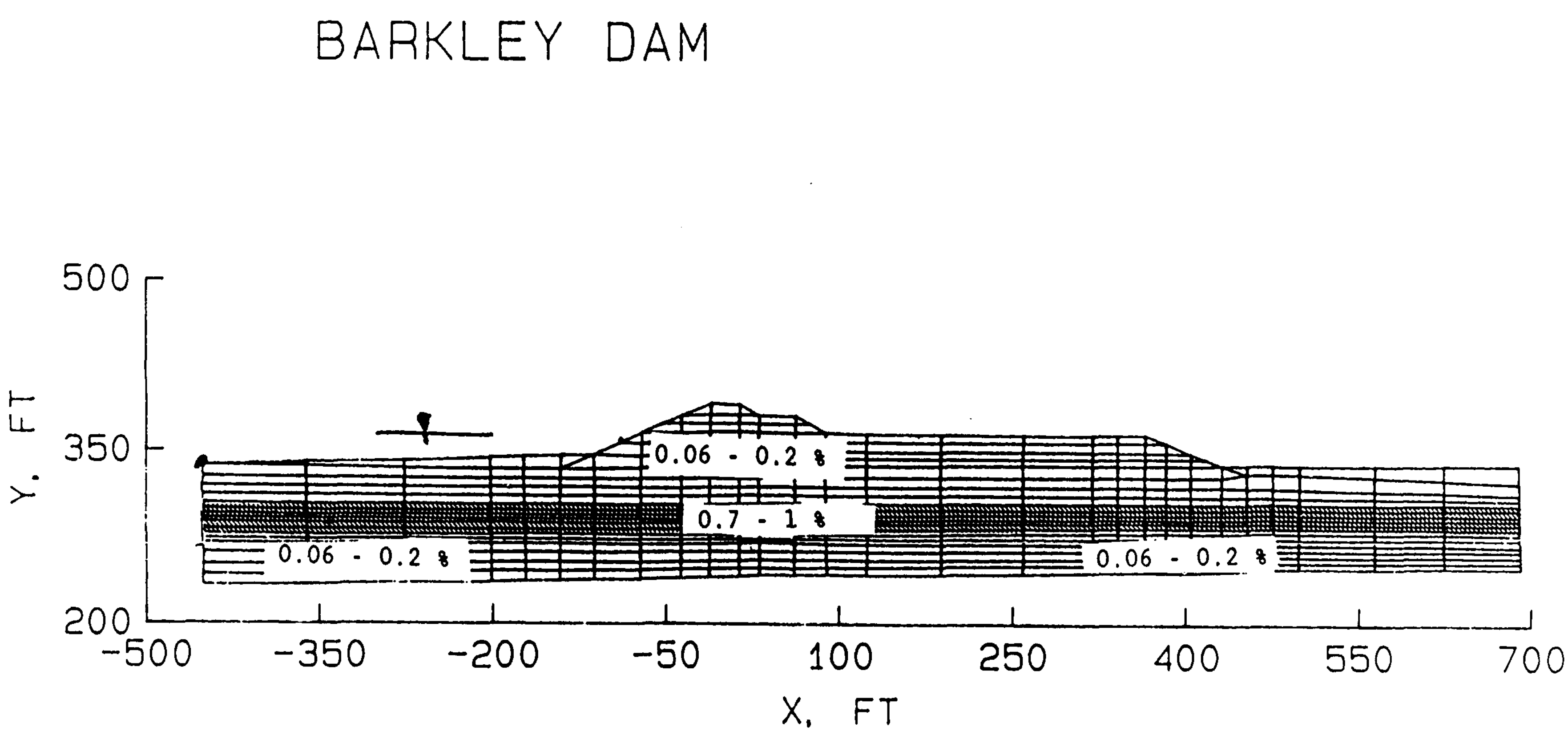
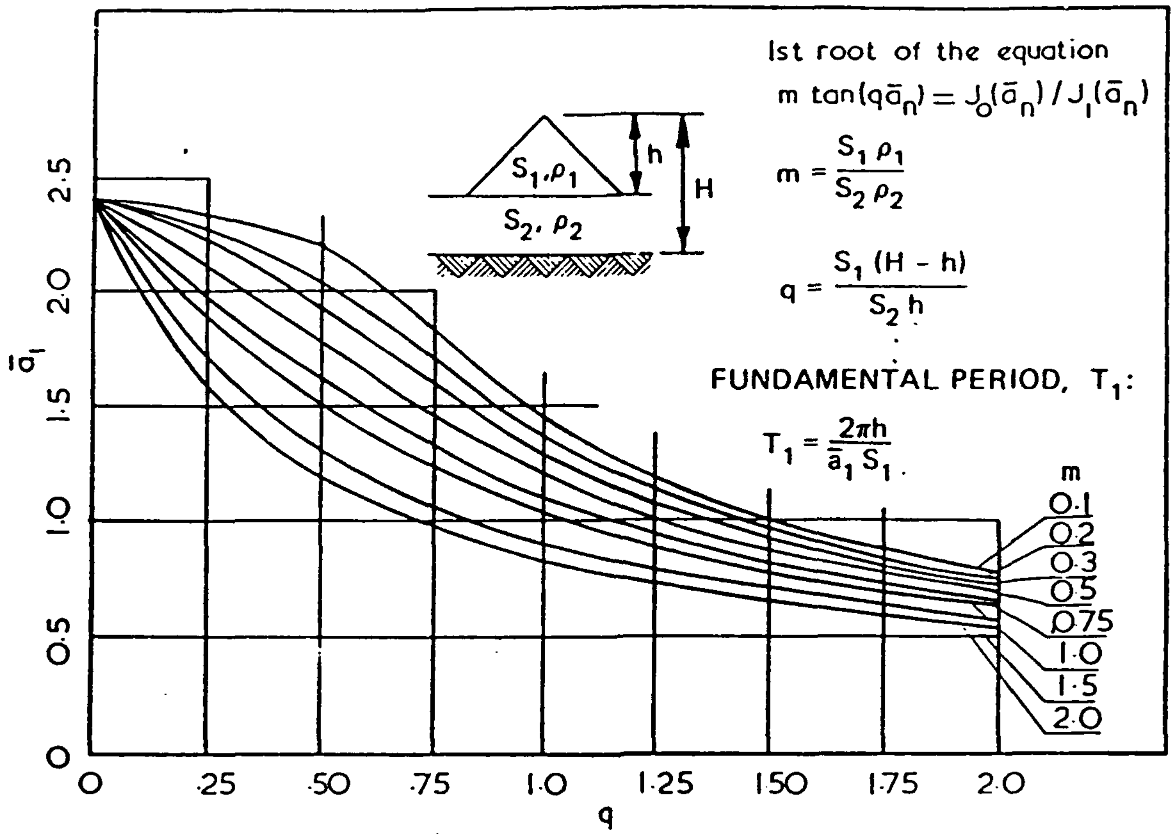


Figure 51. Effective earthquake induced strains.



Properties and Dimensions for Barkley Dam:

h = 50 ft	ρ_1 = 125 pcf	S_1 = 600 fps
H = 175 ft	ρ_2 = 125 pcf	S_2 = 800 fps

Compute m and q:

m = (600 * 125)/(800 *125) = 0.75

q = (600 * (175 - 50))/(800 * 50) = 1.875

From chart: a_1 = 0.7

Period: T_1 = (2 * π * 50)/(0.7 * 600)

= 0.75 sec

Figure 52. Estimate of pre-earthquake fundamental period of Barkley Dam.

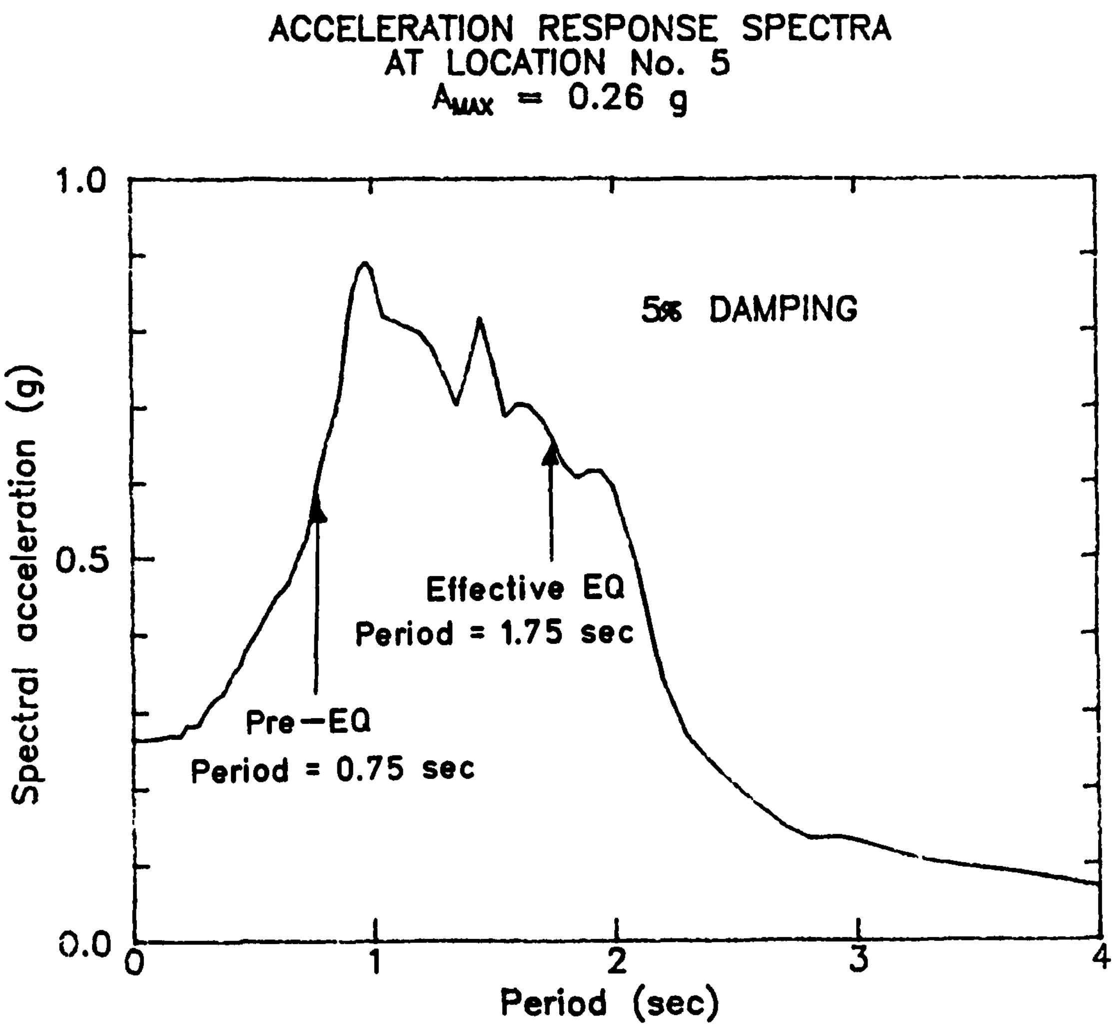


Figure 53. Comparison of pre- and effective earthquake periods with acceleration response spectra of input motions to finite element analysis.

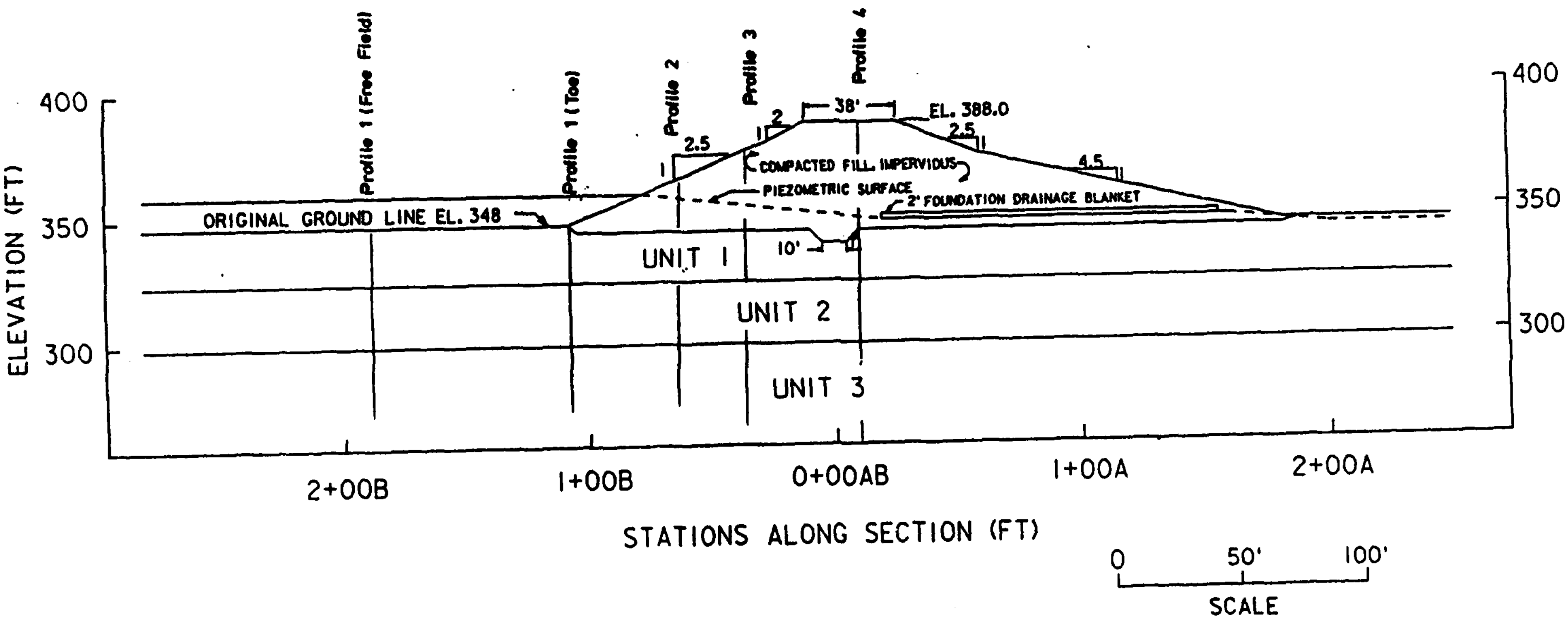


Figure 54. Representative cross section and location of one-dimensional profiles used to approximate dynamic the response of the main embankment.

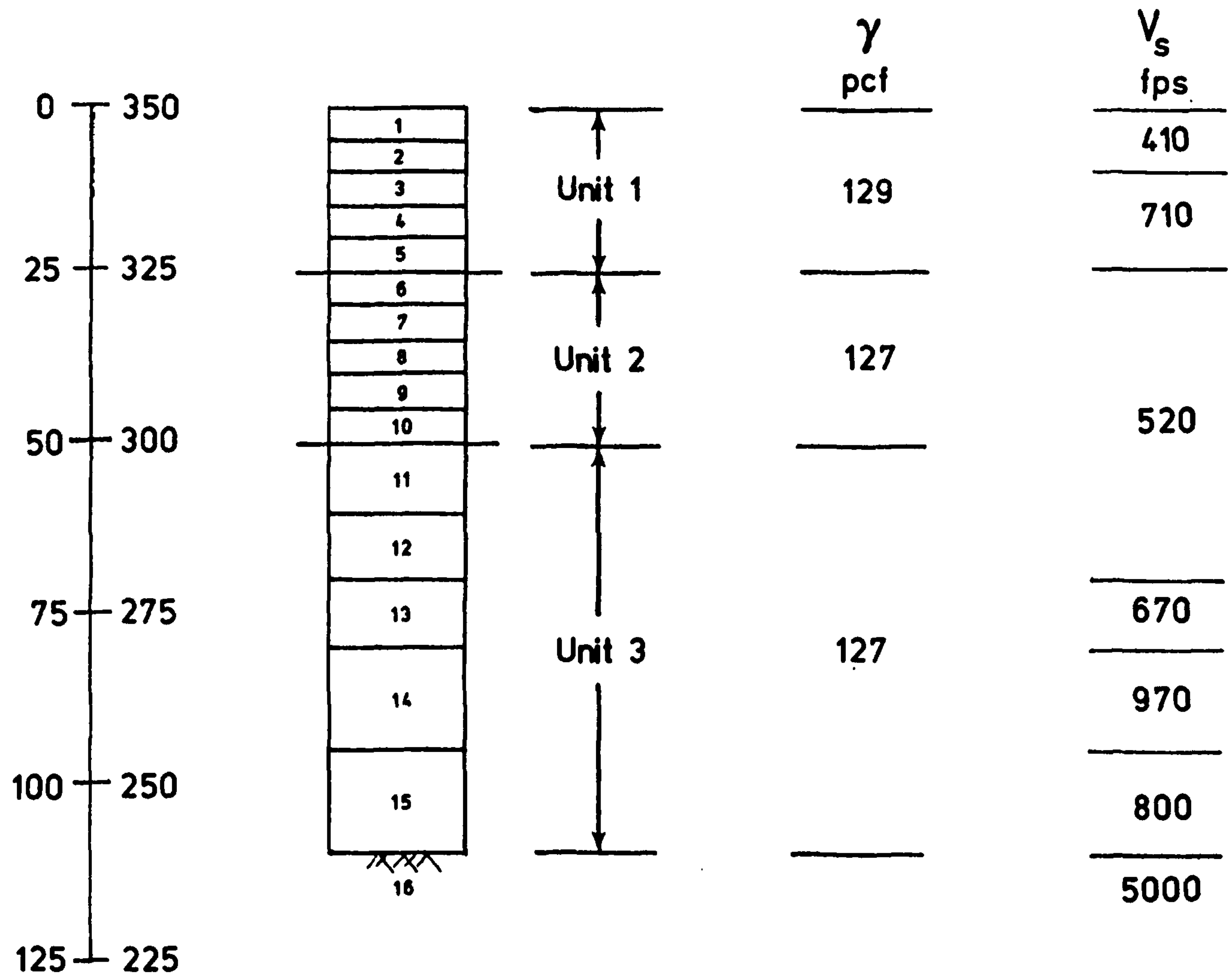


Figure 55. Profile 1 - upstream free field of main embankment.

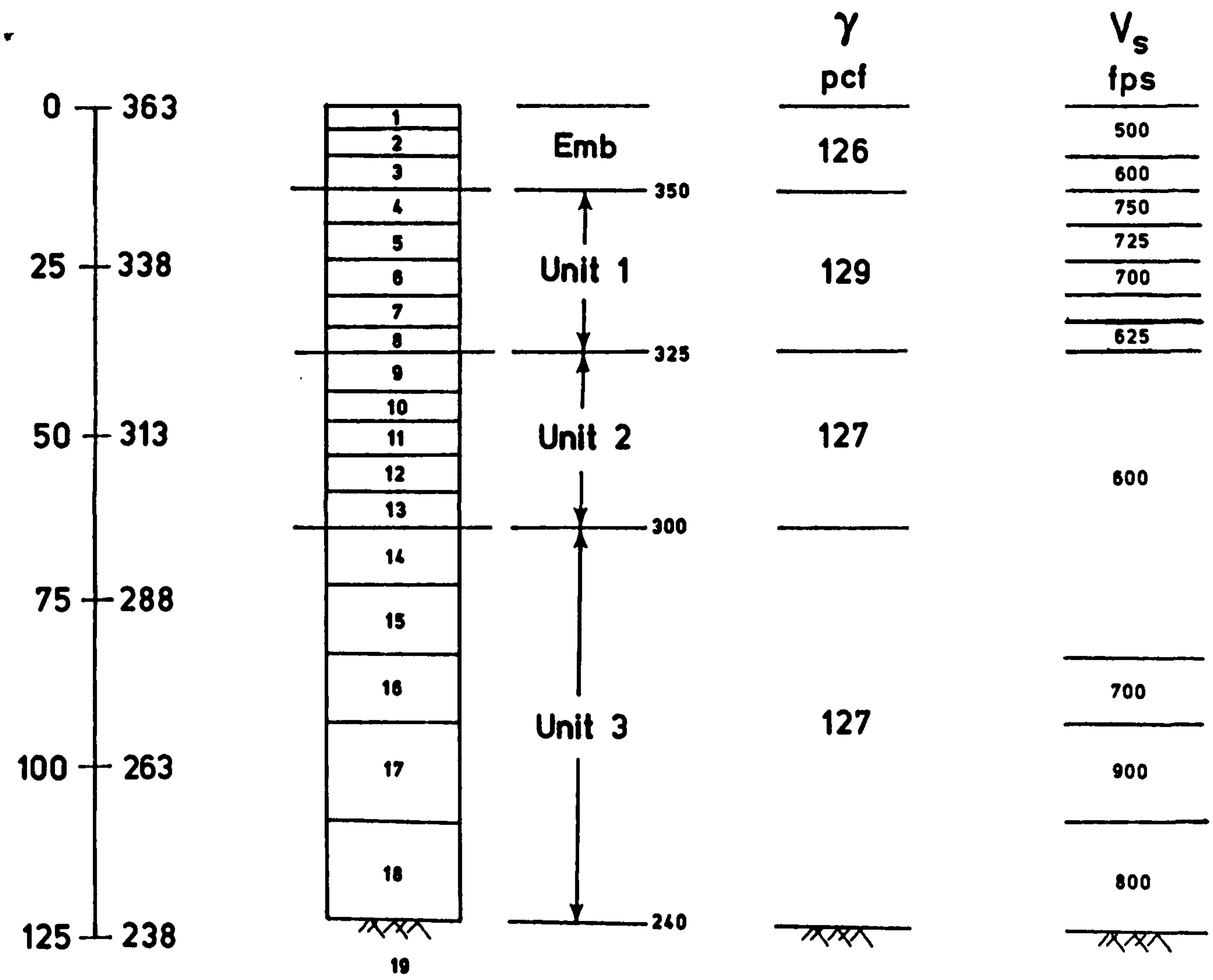


Figure 56. Profile 2 – main embankment profile located at a point one-third of the way up the face of the upstream side of the embankment.

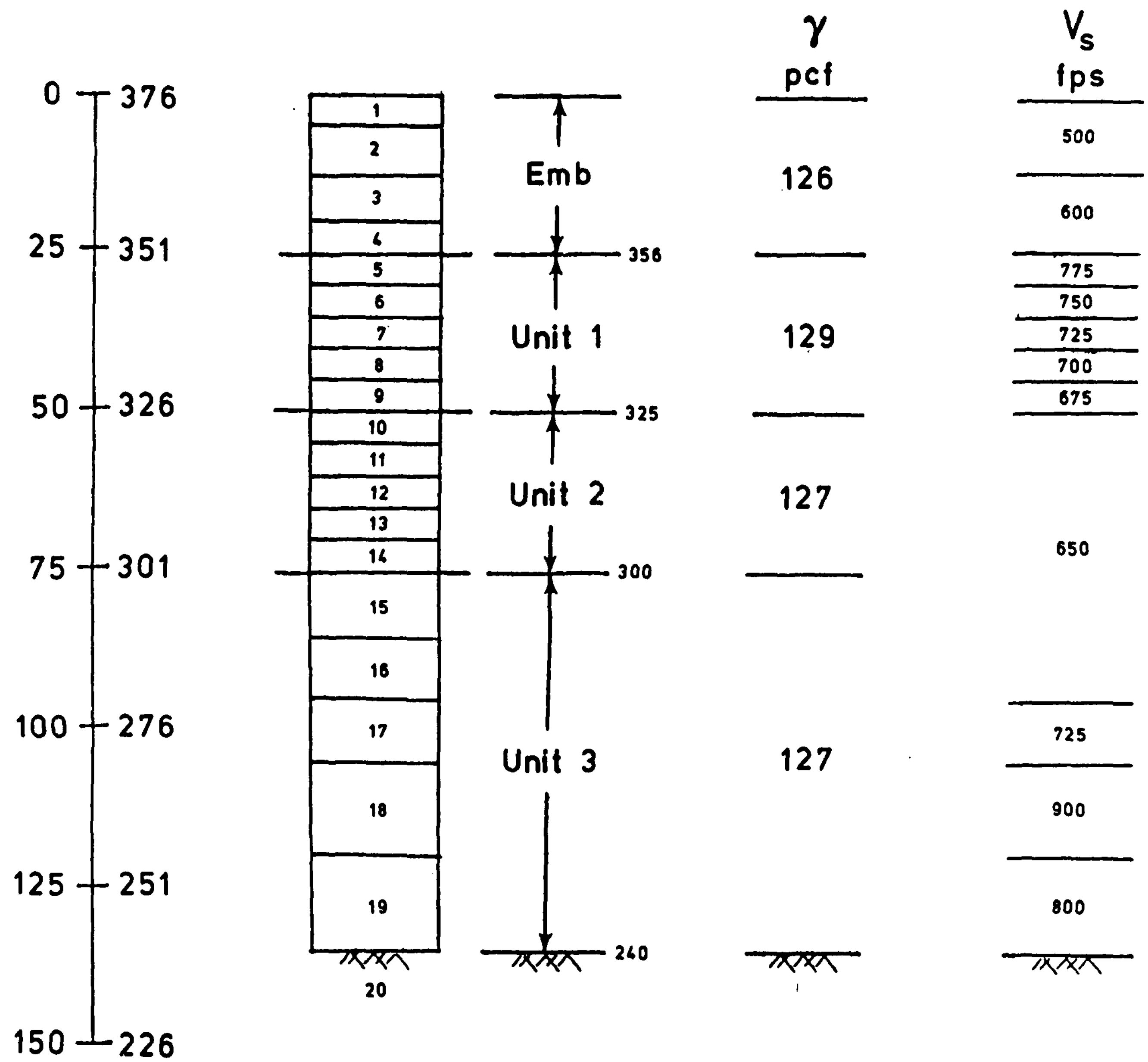


Figure 57. Profile 3 - main embankment profile at a point two-thirds of the way up the face of the face of the upstream side of the embankment.

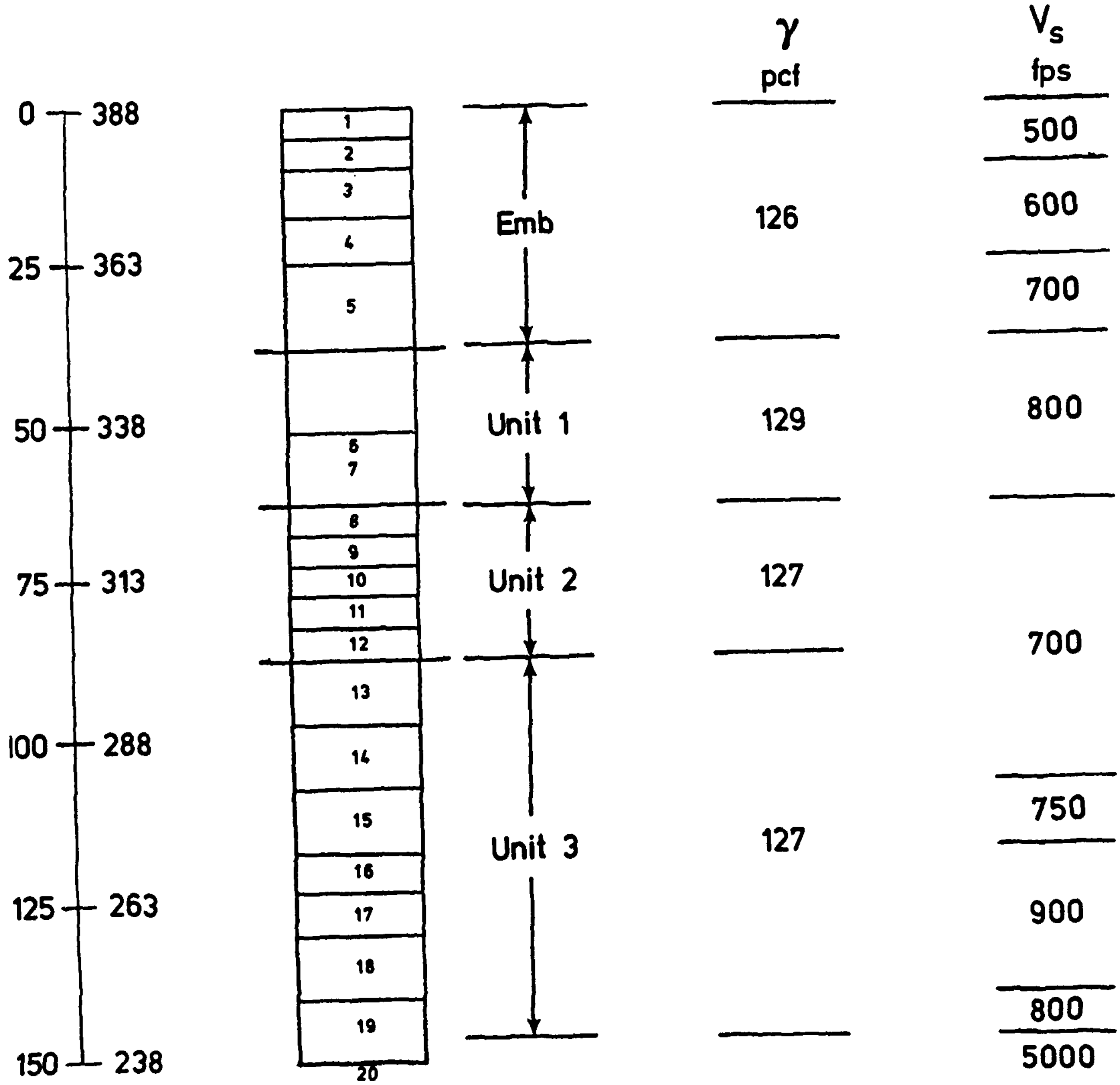


Figure 58. Profile 4 - main embankment profile located at a point on the centerline of the embankment.

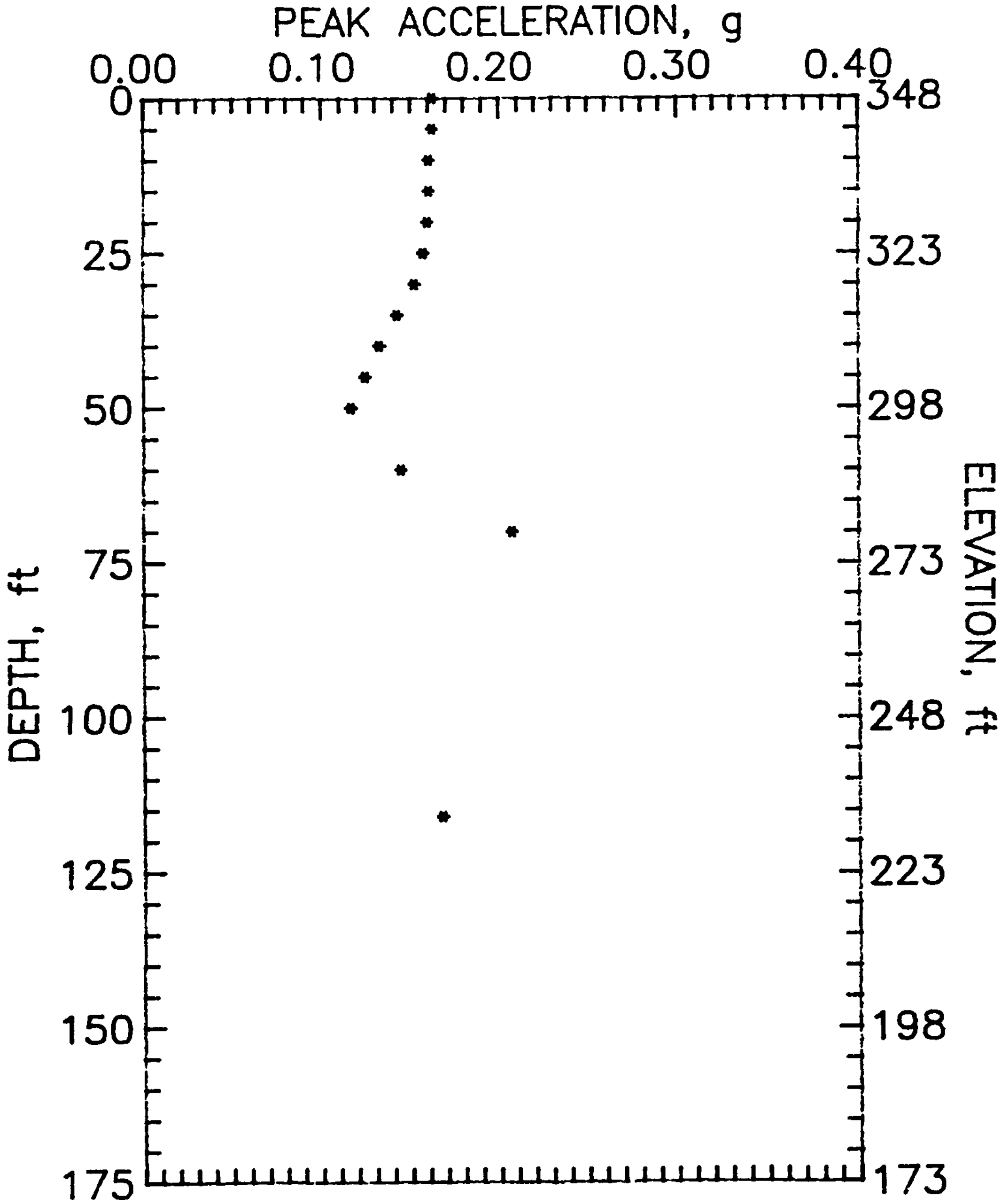


Figure 59. Peak acceleration versus depth for Profile 1 of main embankment.

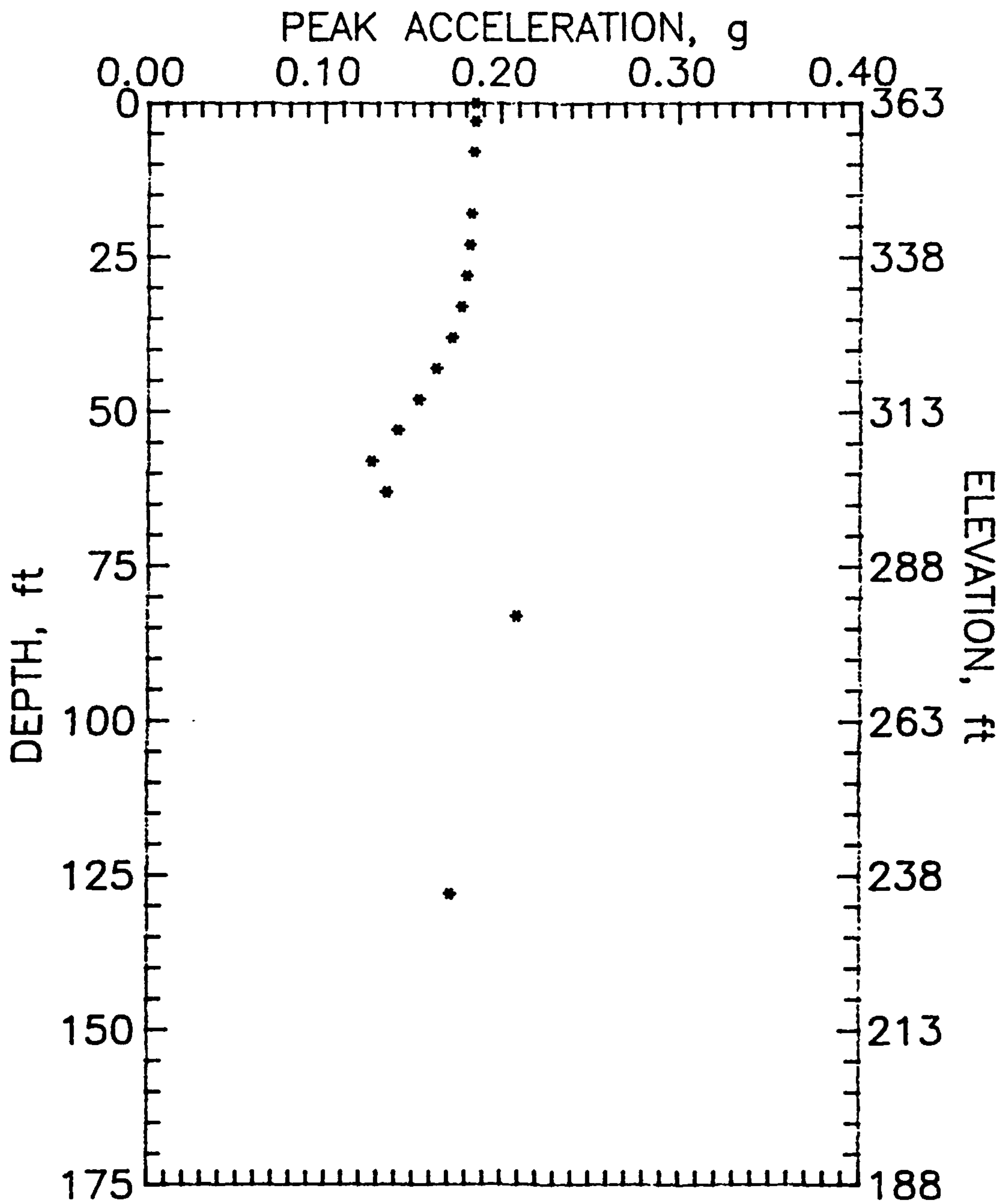


Figure 60. Peak acceleration versus depth for Profile 2 of main embankment.

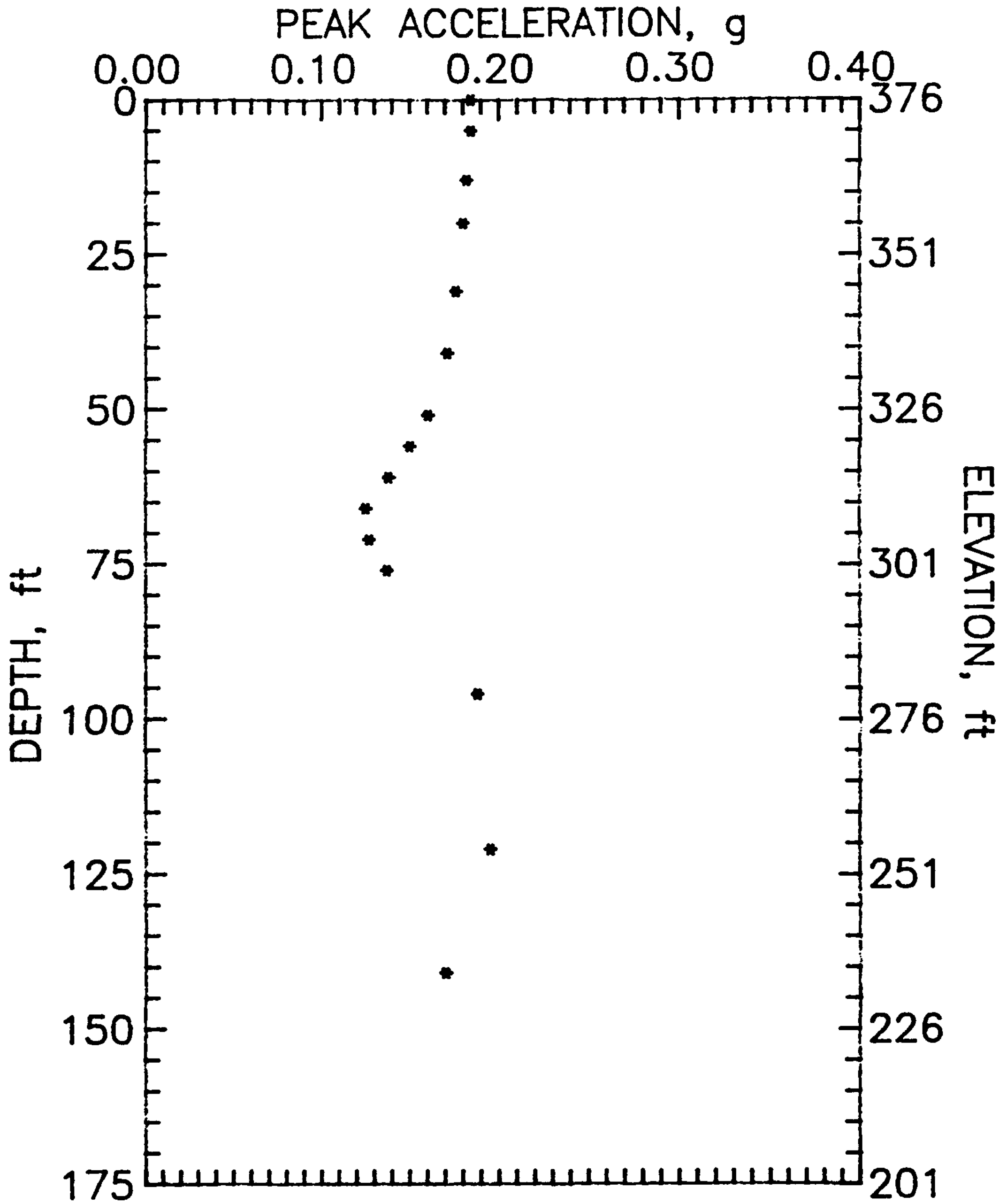


Figure 61. Peak acceleration versus depth for Profile 3 of main embankment.

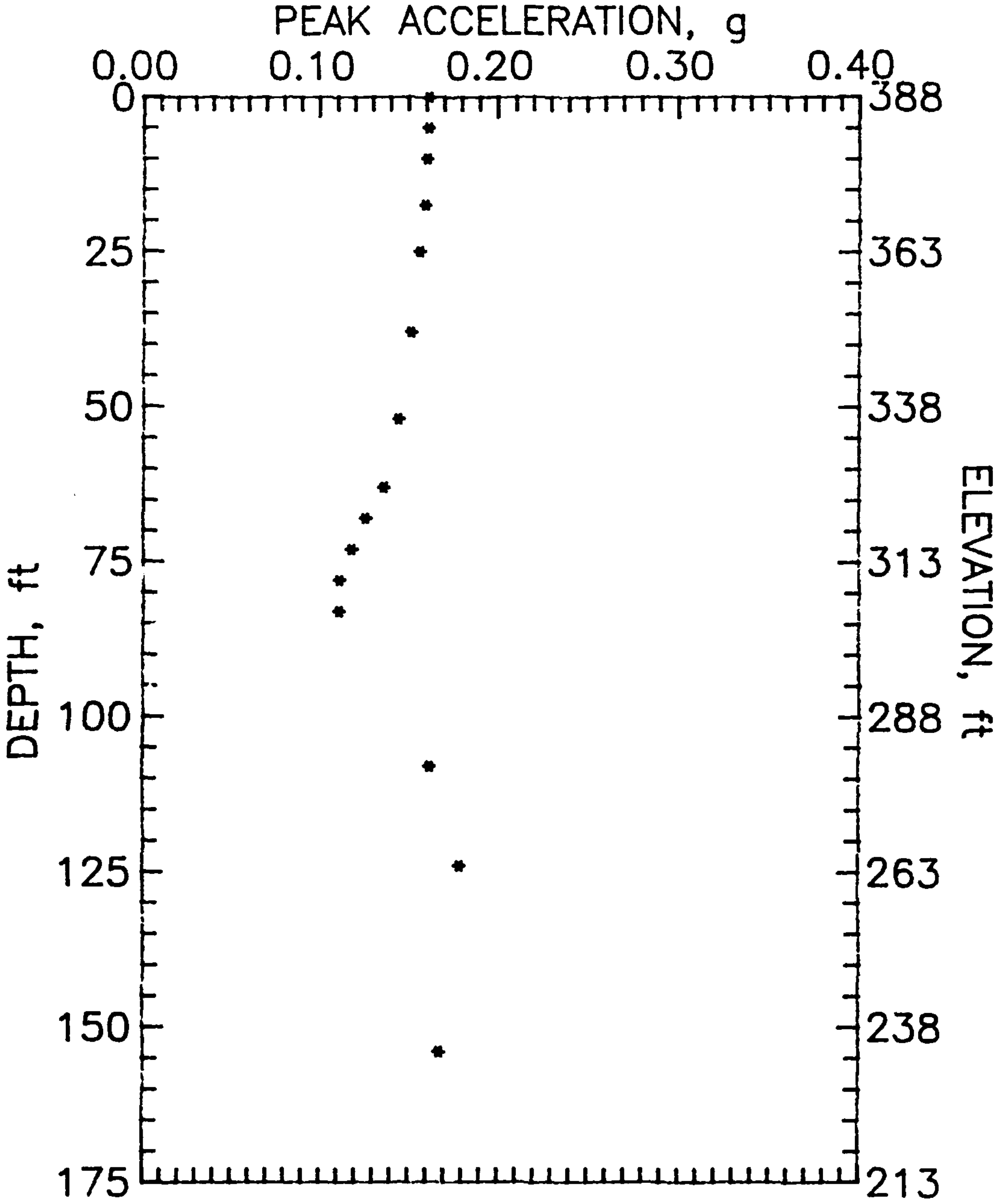


Figure 62. Peak acceleration versus depth for Profile 4 of main embankment.

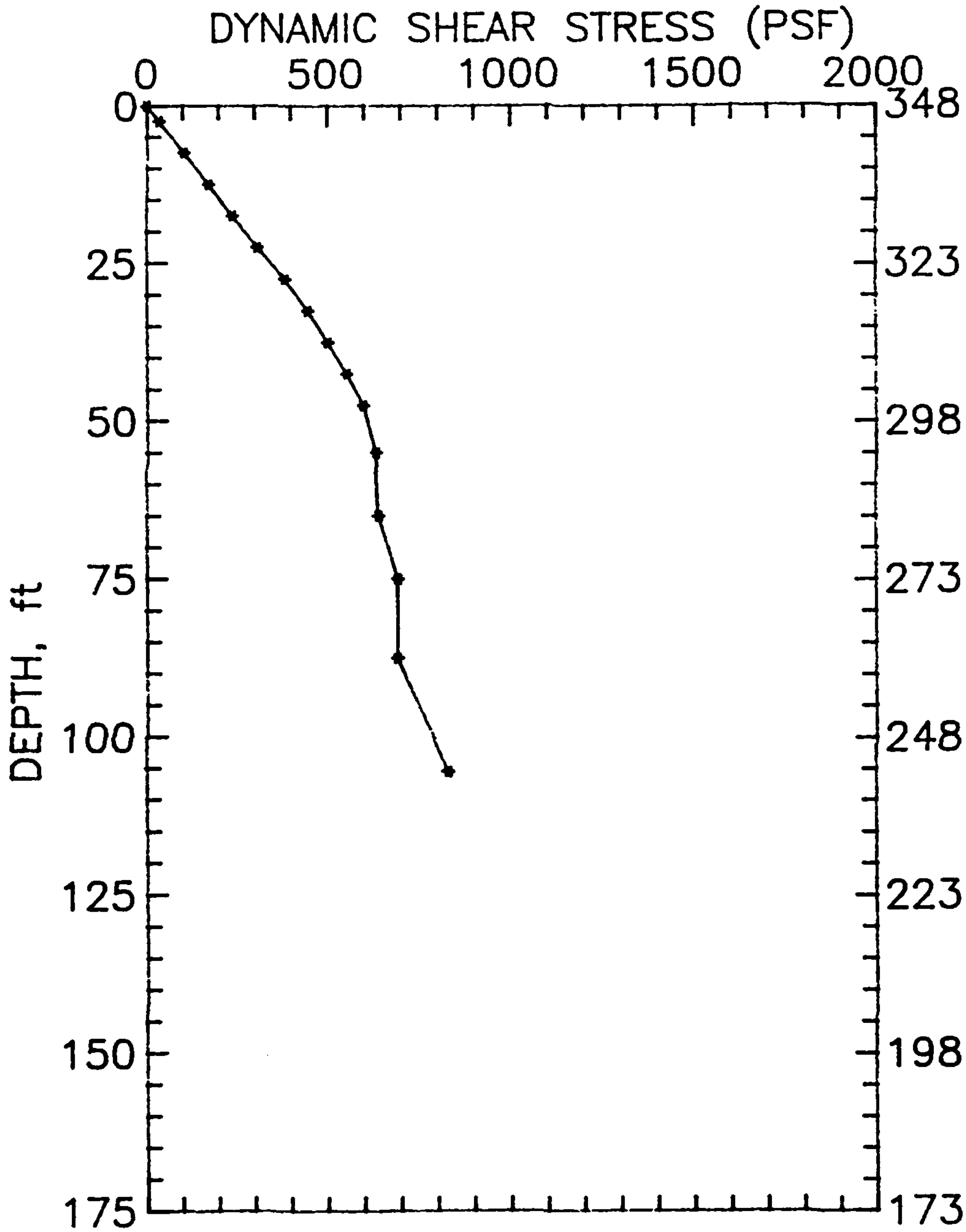


Figure 63. Dynamic shear stresses versus depth in Profile 1.

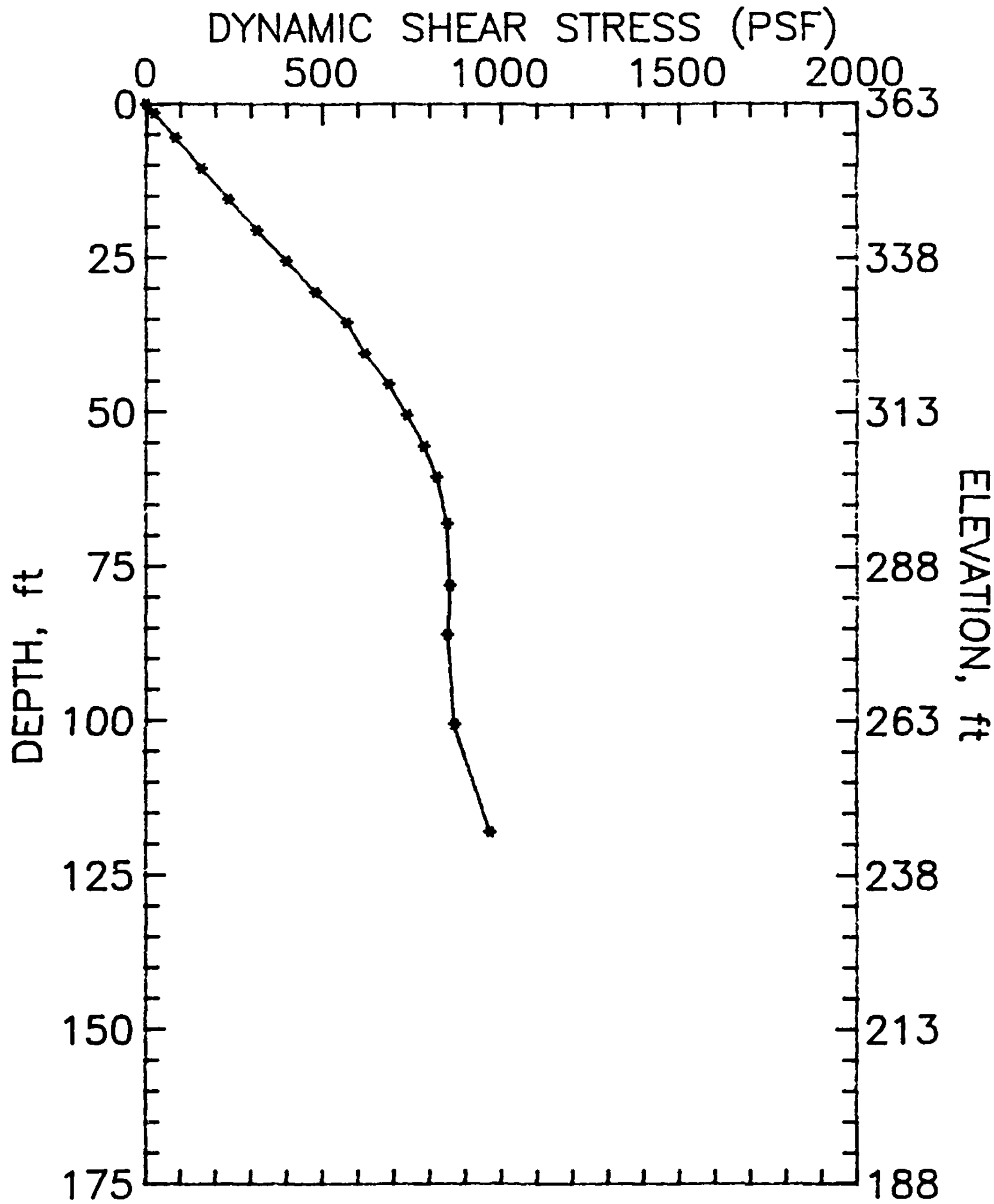


Figure 64. Dynamic shear stresses versus depth in Profile 2.

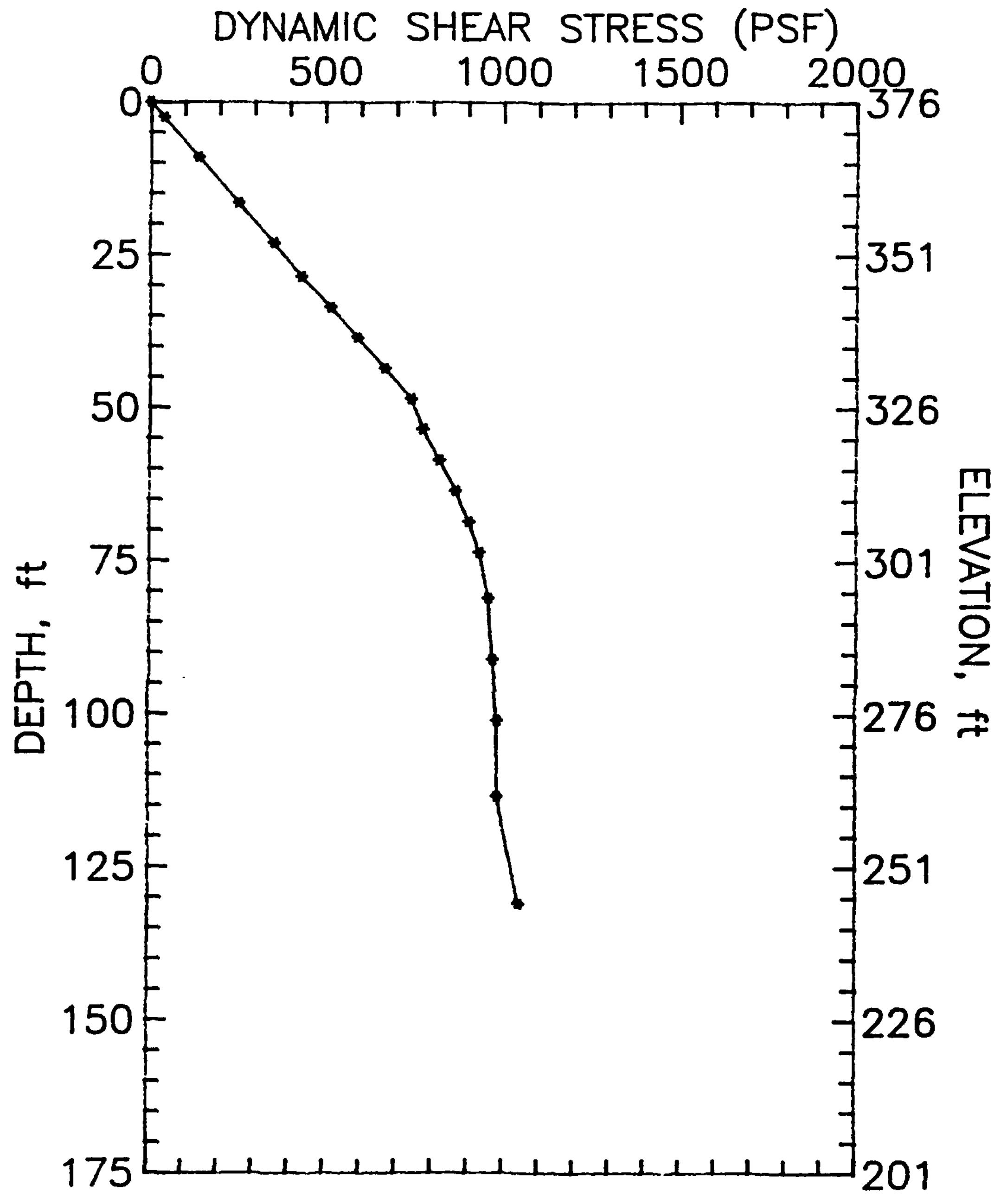


Figure 65. Dynamic shear stresses versus depth in Profile 3.

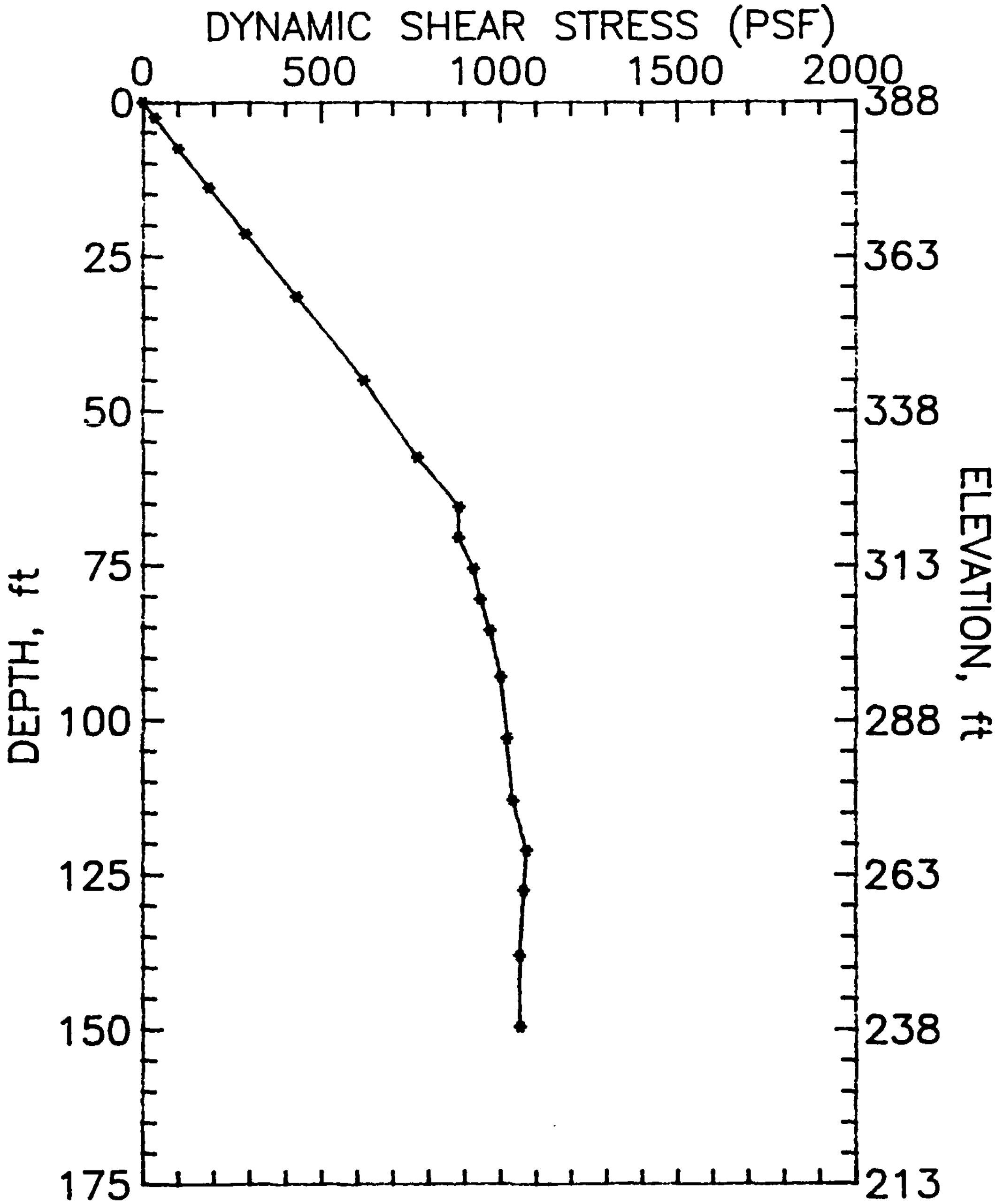


Figure 66. Dynamic shear stresses versus depth in Profile 4.

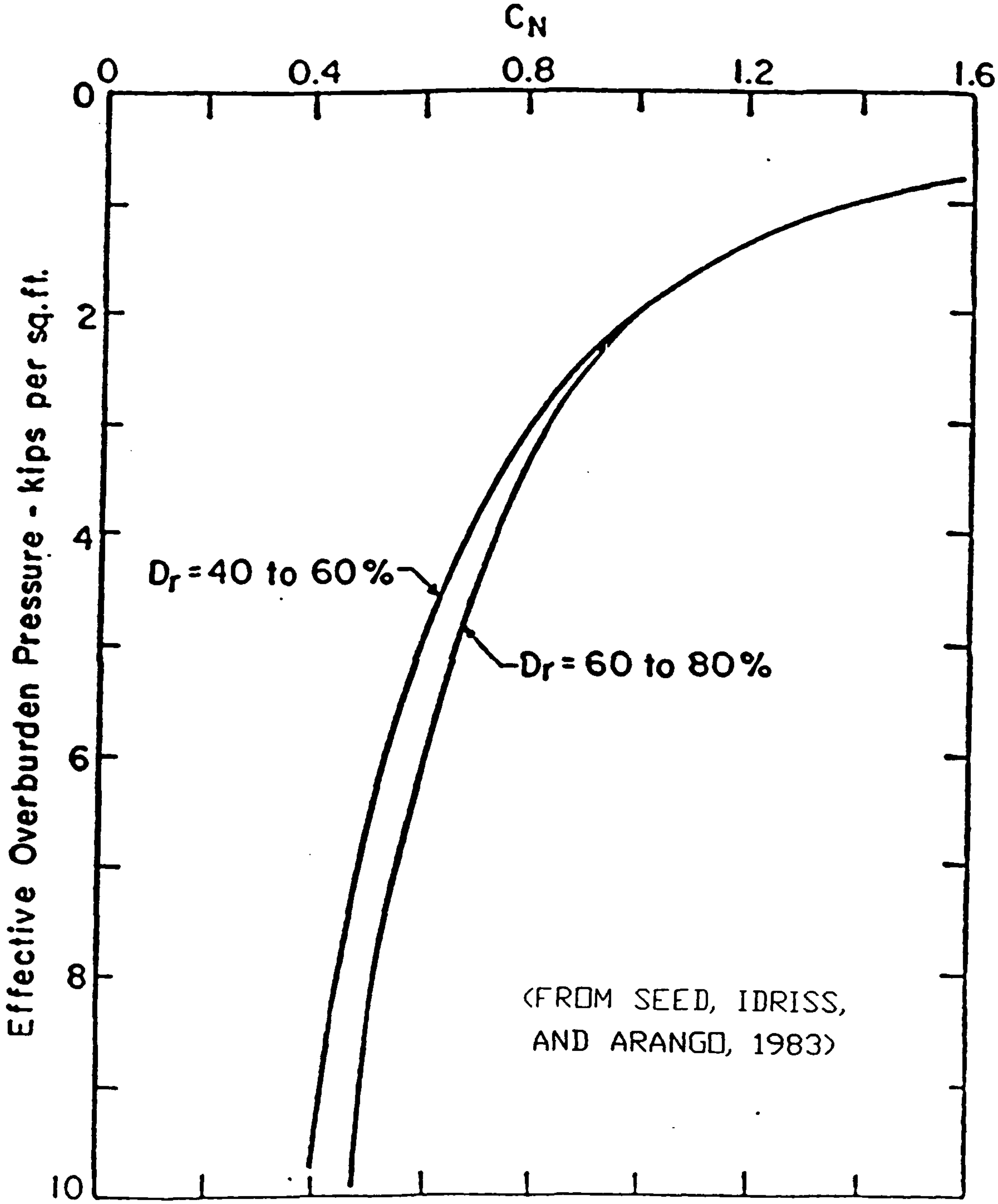


Figure 67. Correction factor, Cn, to determine the equivalent blowcounts, N_1, at confining stresses of 1 TSF.

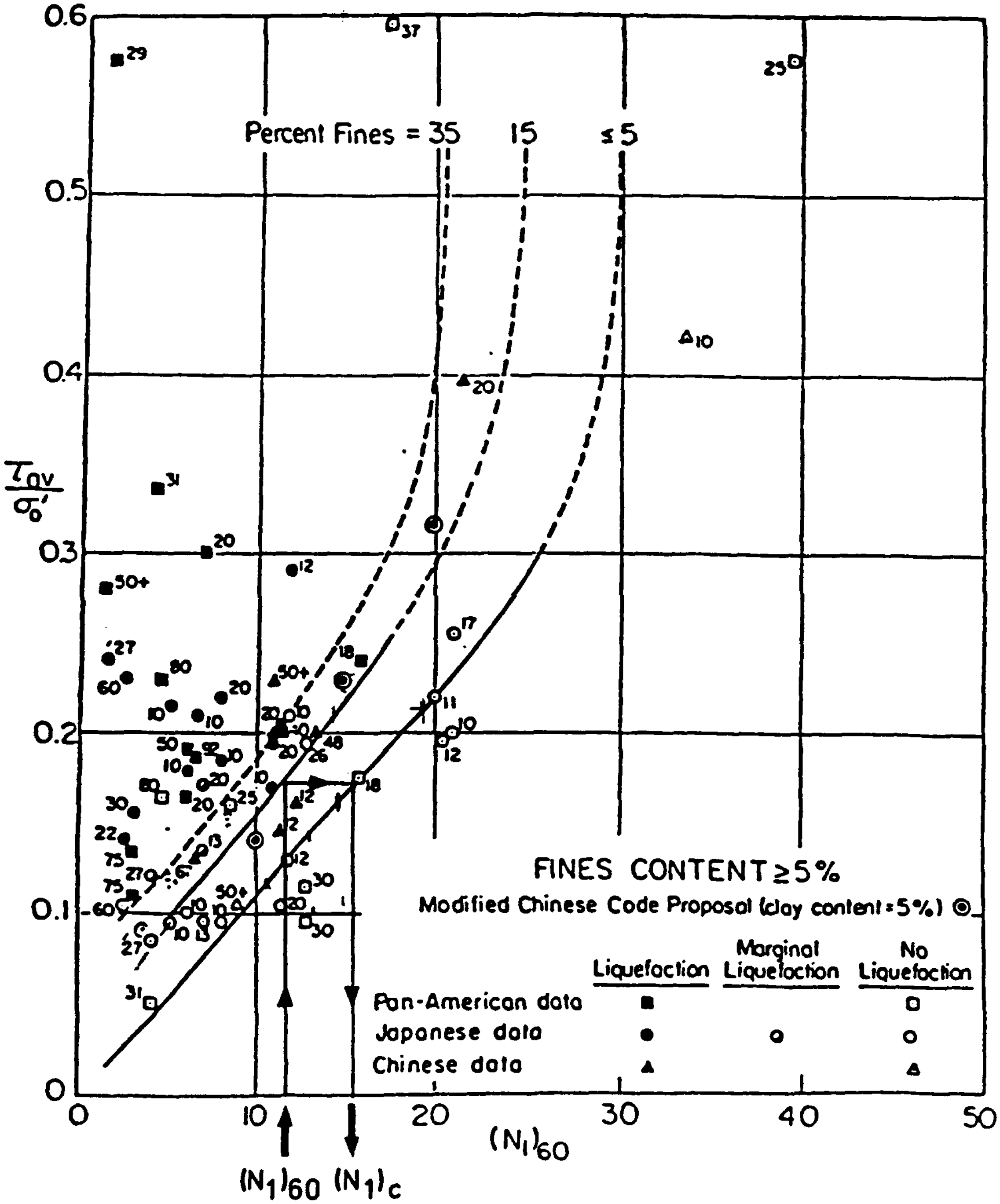


Figure 68. Empirical Chart for liquefaction resistance of Sands and Silty Sands for local magnitude 7.5 earthquakes (from Seed, Tokimatsu, Harder, and Chung, 1984.)

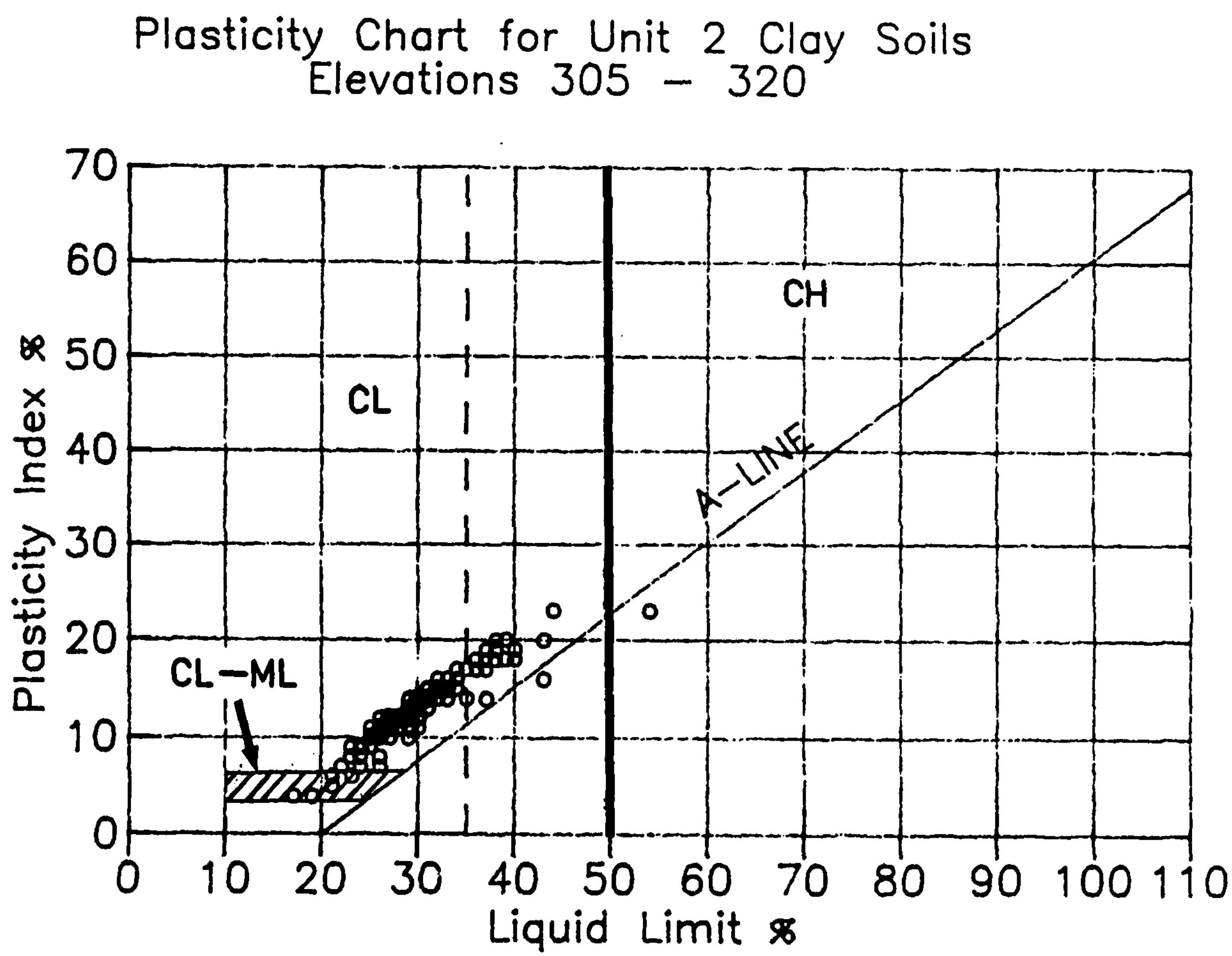


Figure 69. Plasticity Chart for Unit 2 Clay Soils.
(Elevation 305-320)

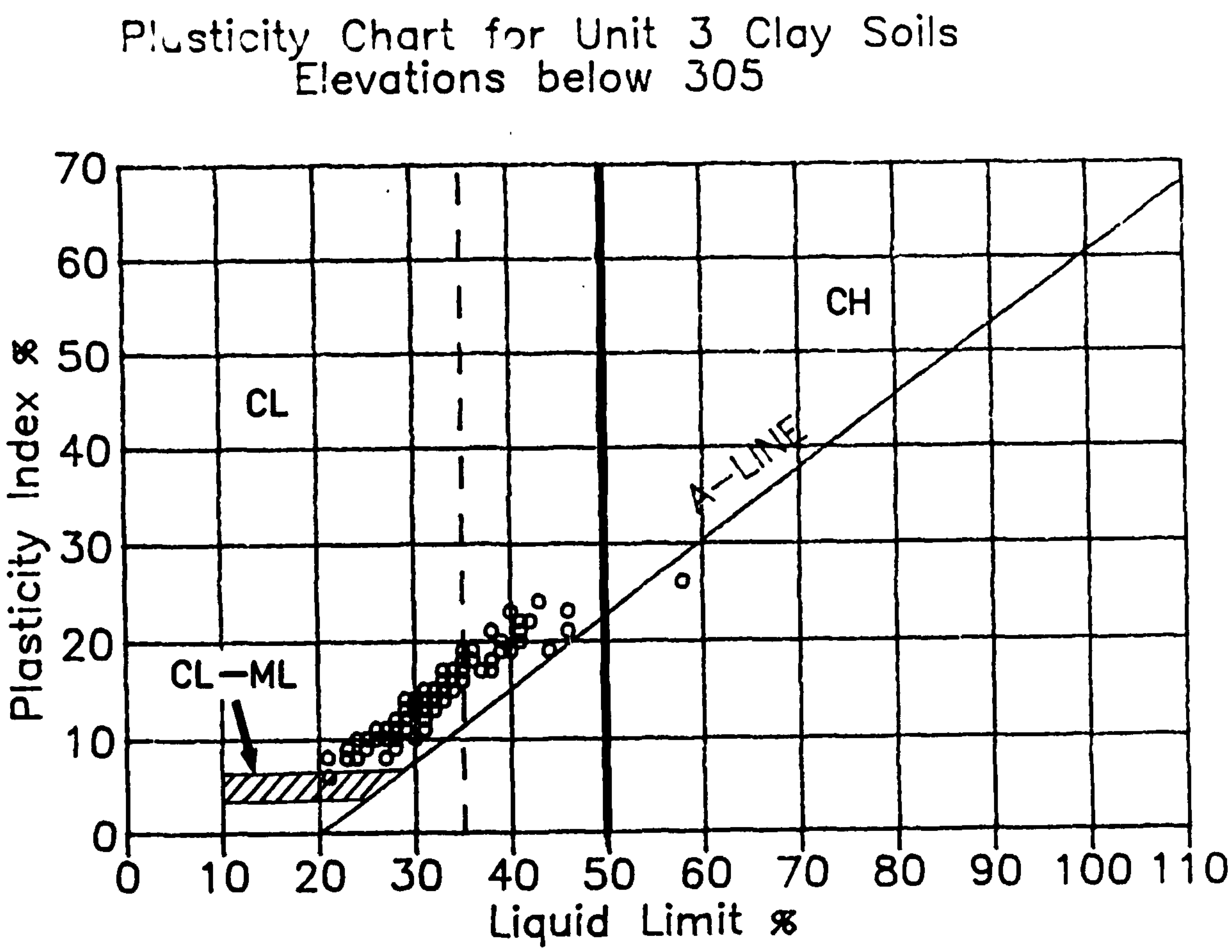


Figure 70. Plasticity Chart for Unit 3 Clay Soils.
(Elevations below 305)

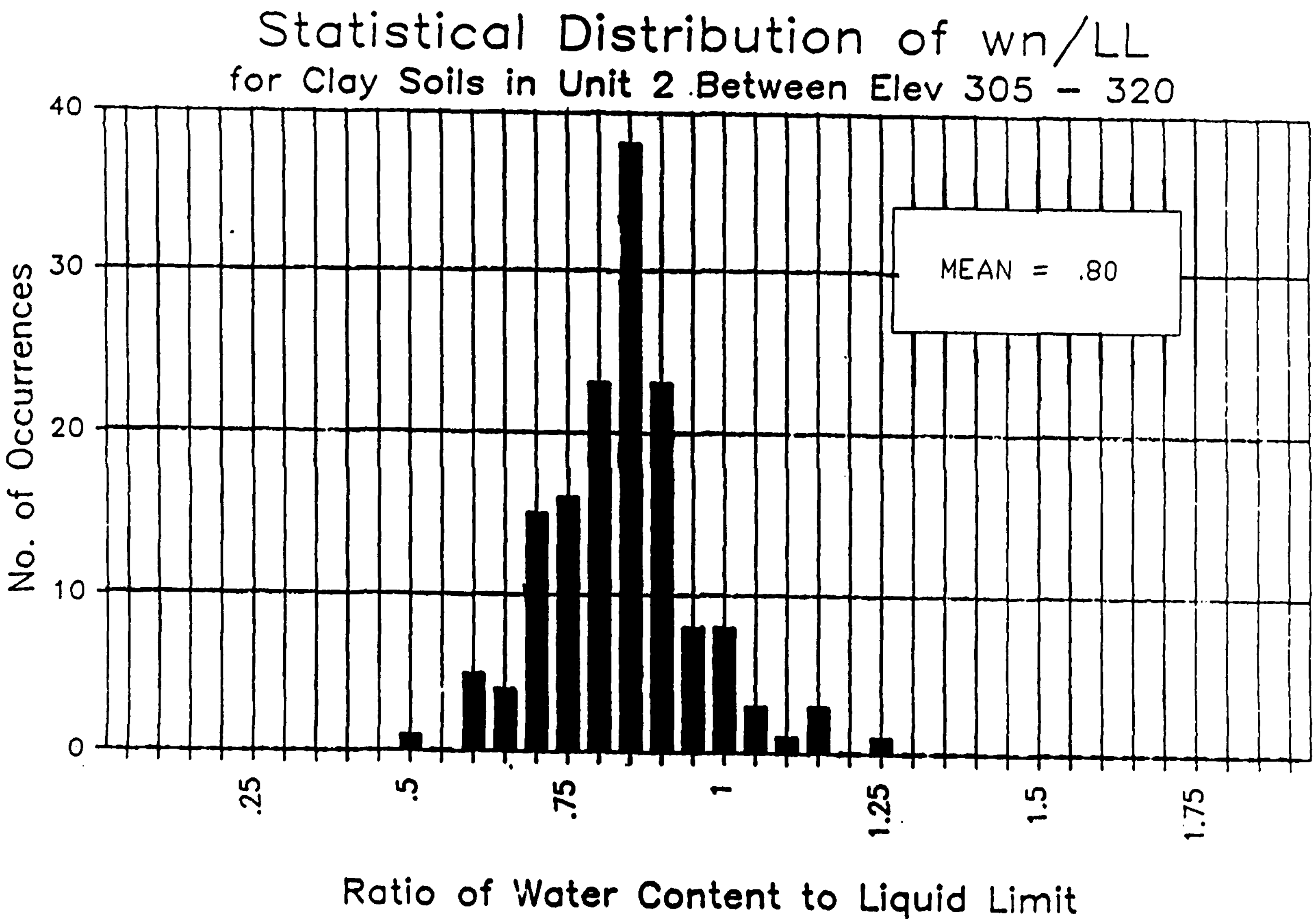


Figure 71. Statistical distribution of the ratio of water content to liquid limit.

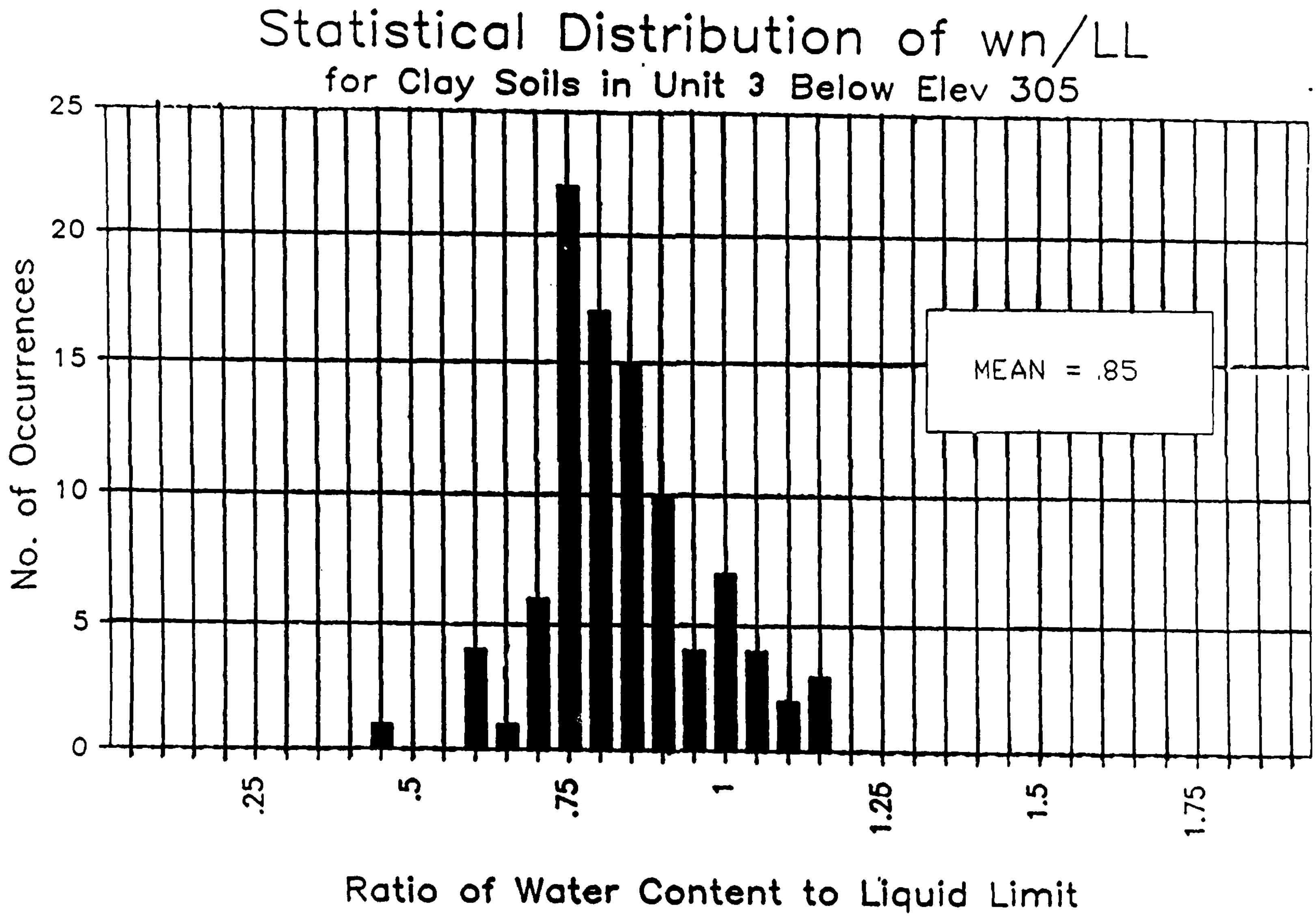


Figure 72. Statistical distribution of the ratio of water content to liquid limit.

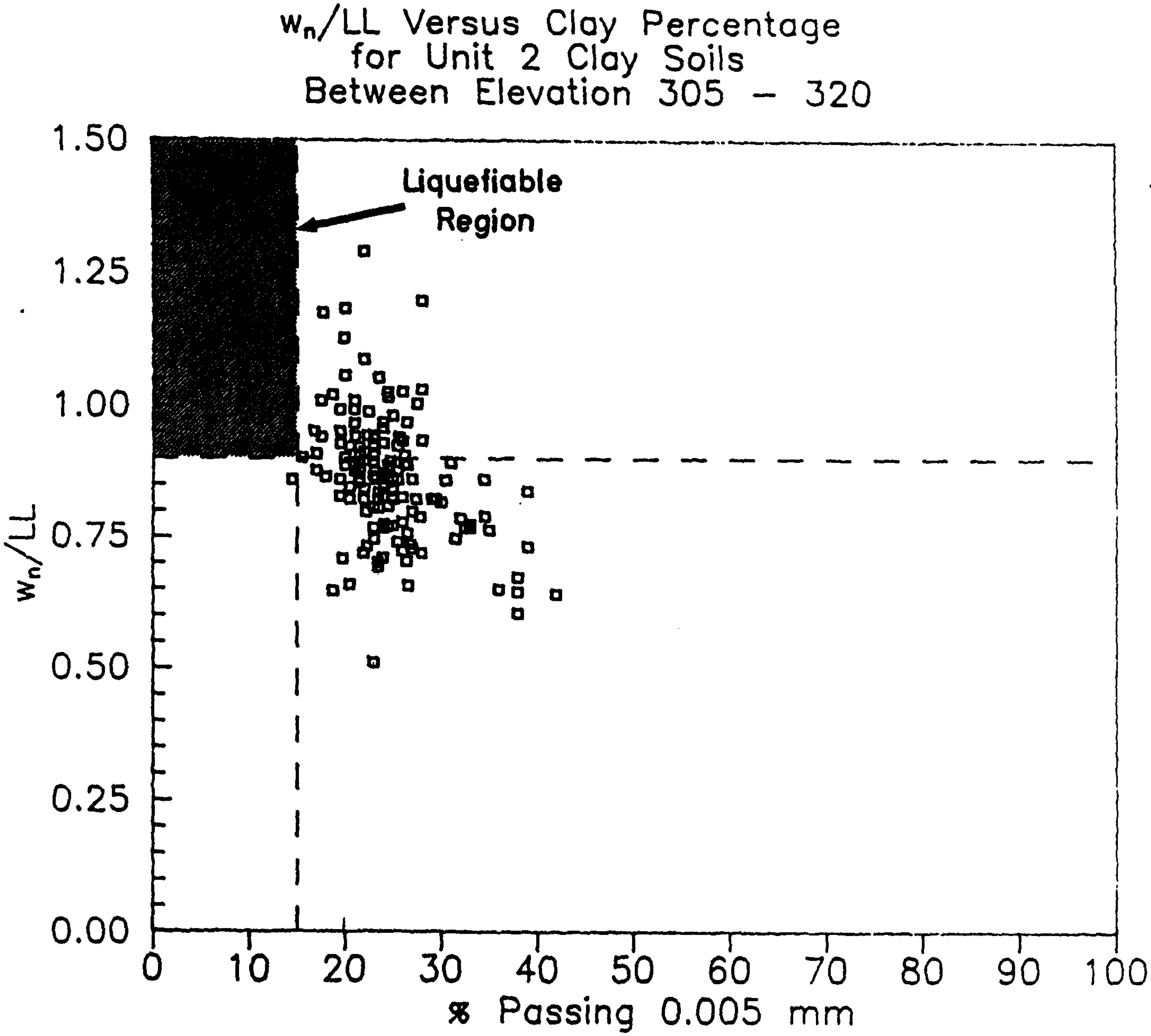


Figure 73. Water content to liquid limit ratio versus percentage passing the 0.005 mm sieve for the Clay *Soils* of Unit 2.

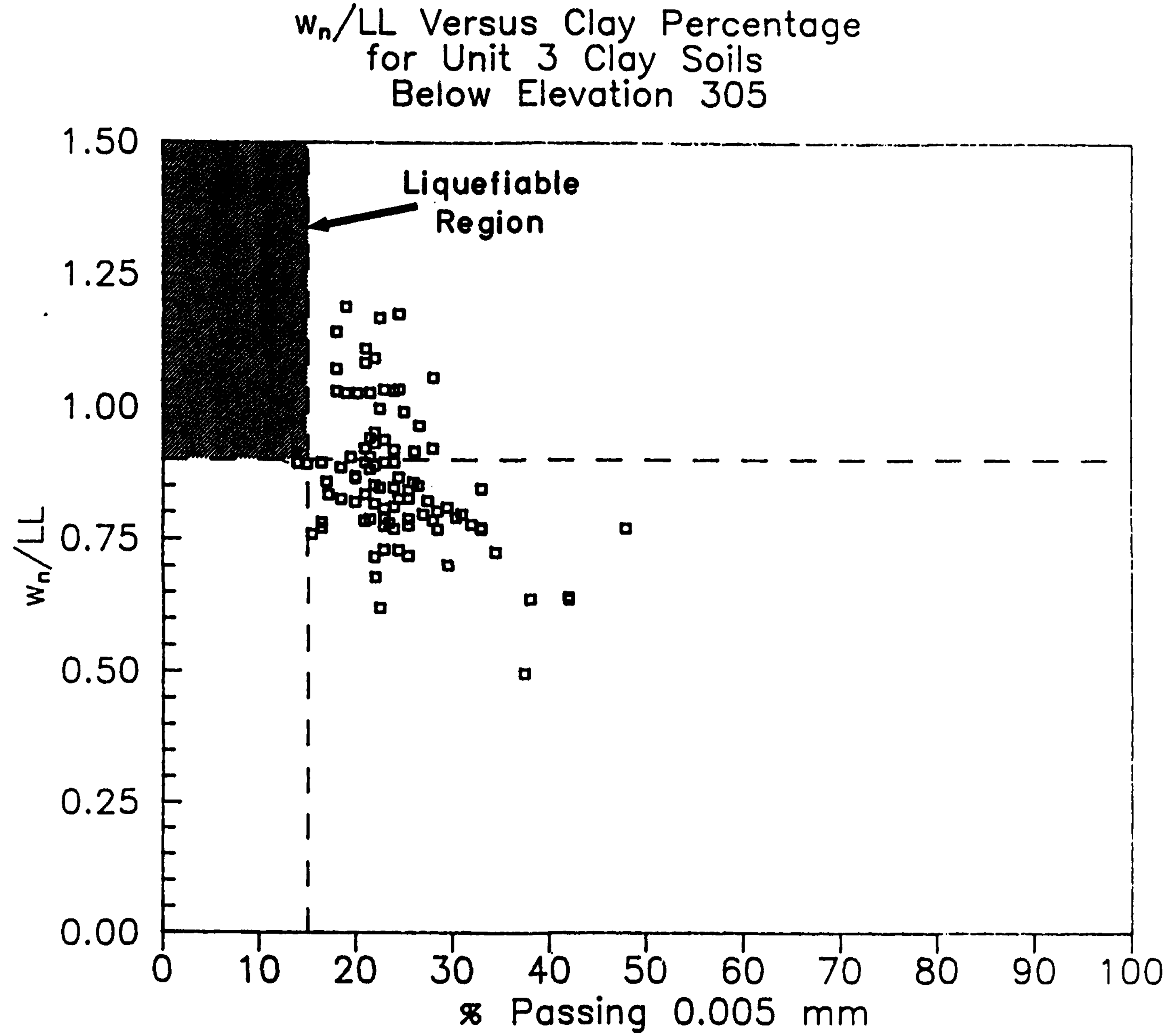


Figure 74. Water content to liquid limit ratio versus percentage passing the 0.005 mm for Clay Soils of Unit 3.

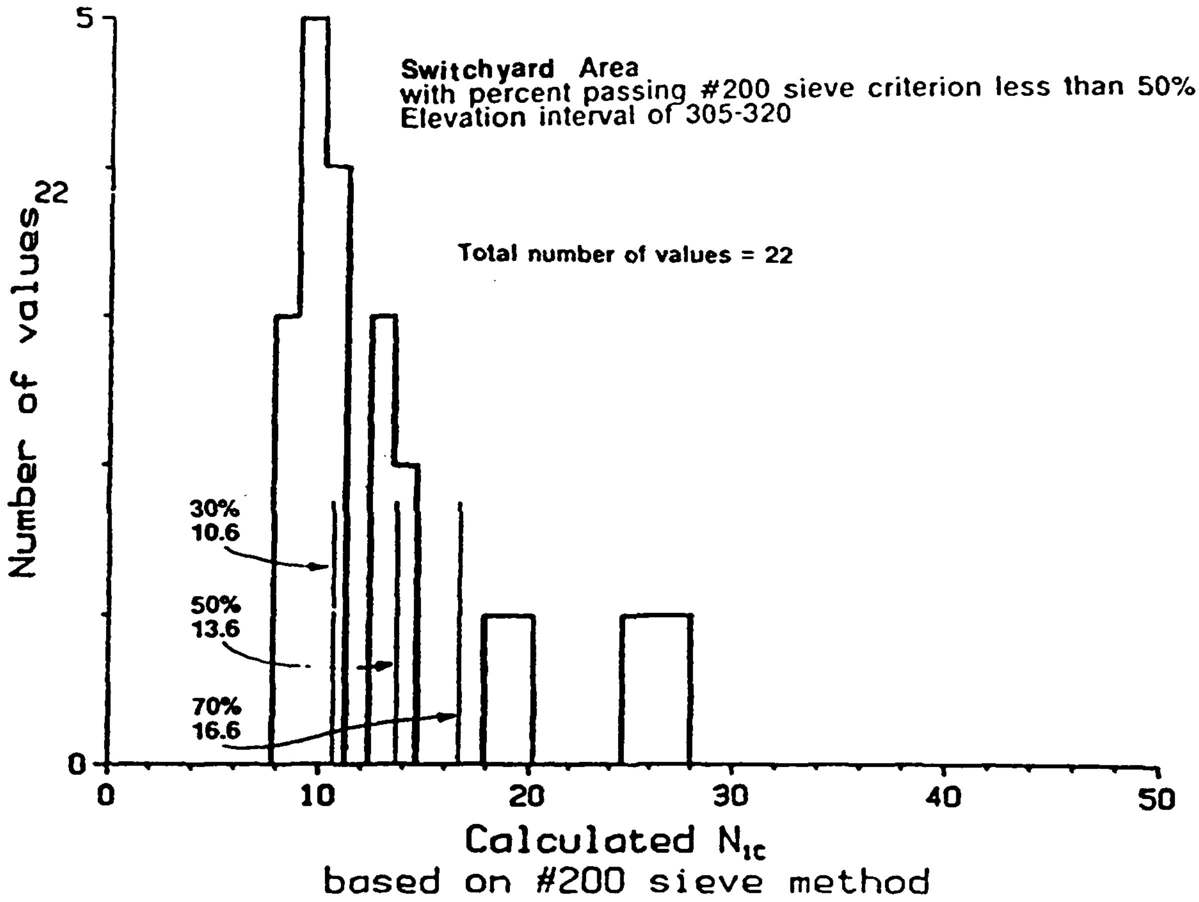


Figure 75 Statistical distribution of measured SPT equivalent clean sand blowcounts in Switchyard Area for Unit 2.

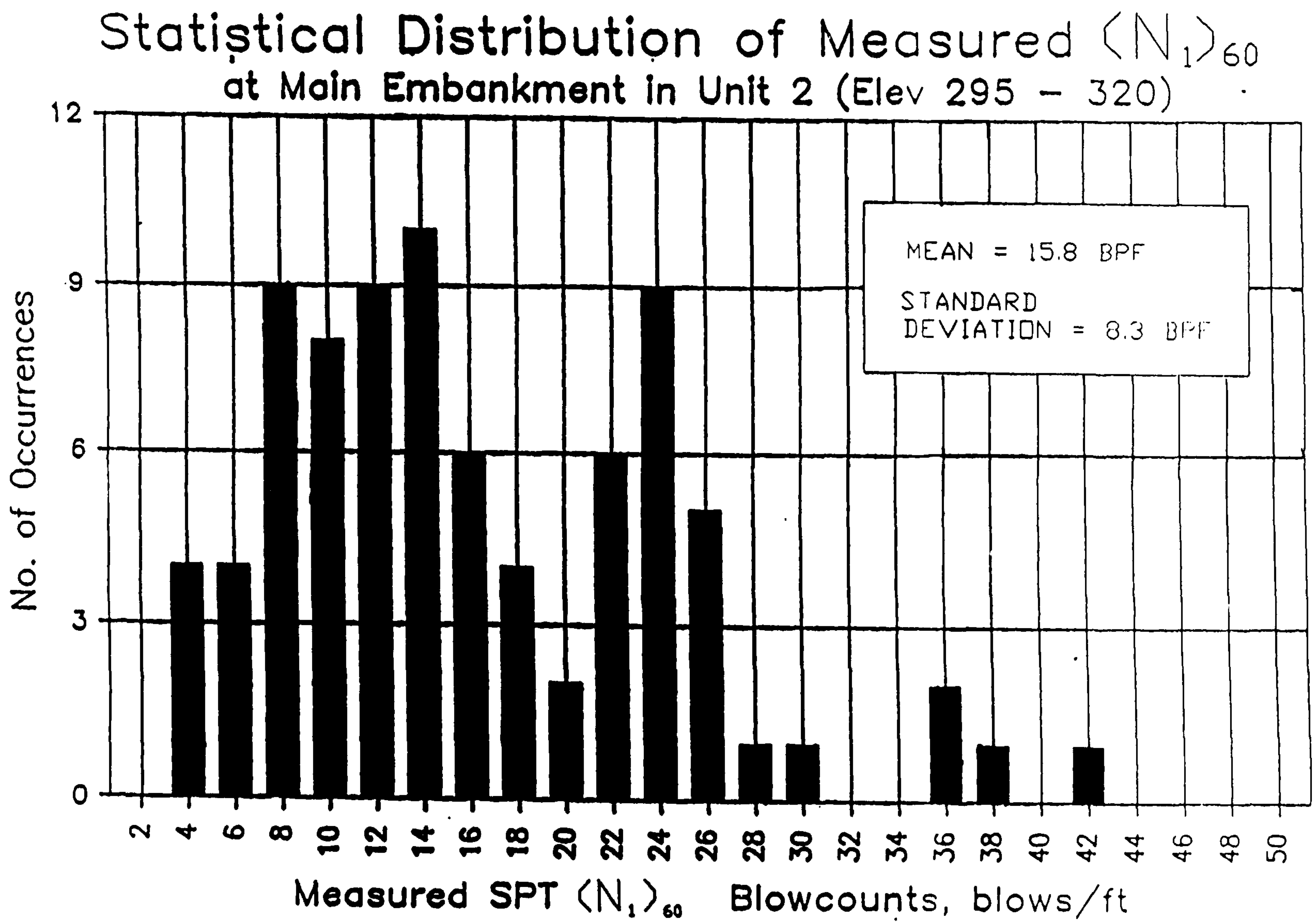


Figure 76. Statistical distribution of the overburden corrected blowcounts, $(N_1)_{60}$, of the main embankment area of Unit 2 between elevations 295 and 320.

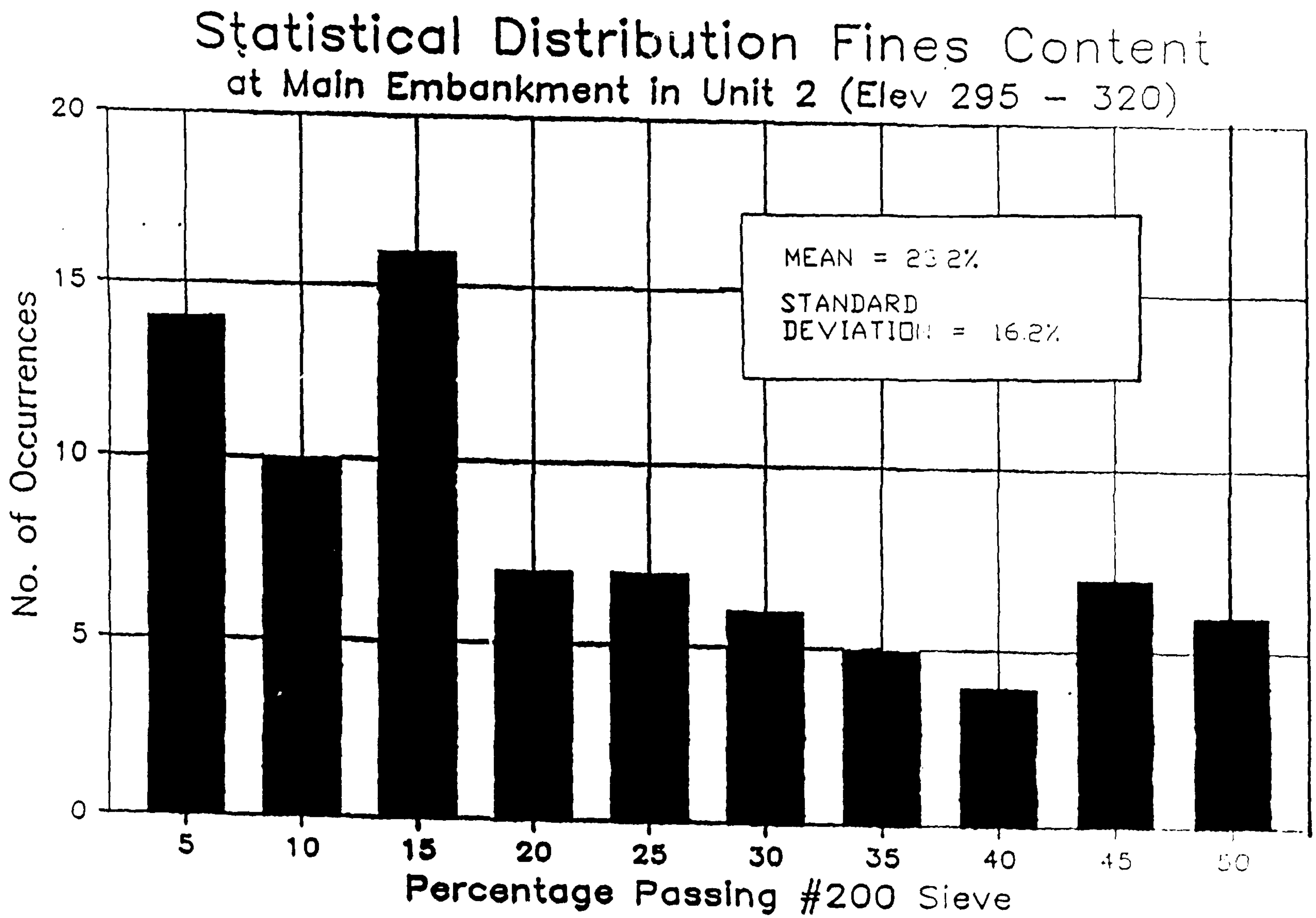


Figure 77. Statistical distribution of the fines content of the main embankment area of Unit 2 between elevations 295 and 320.

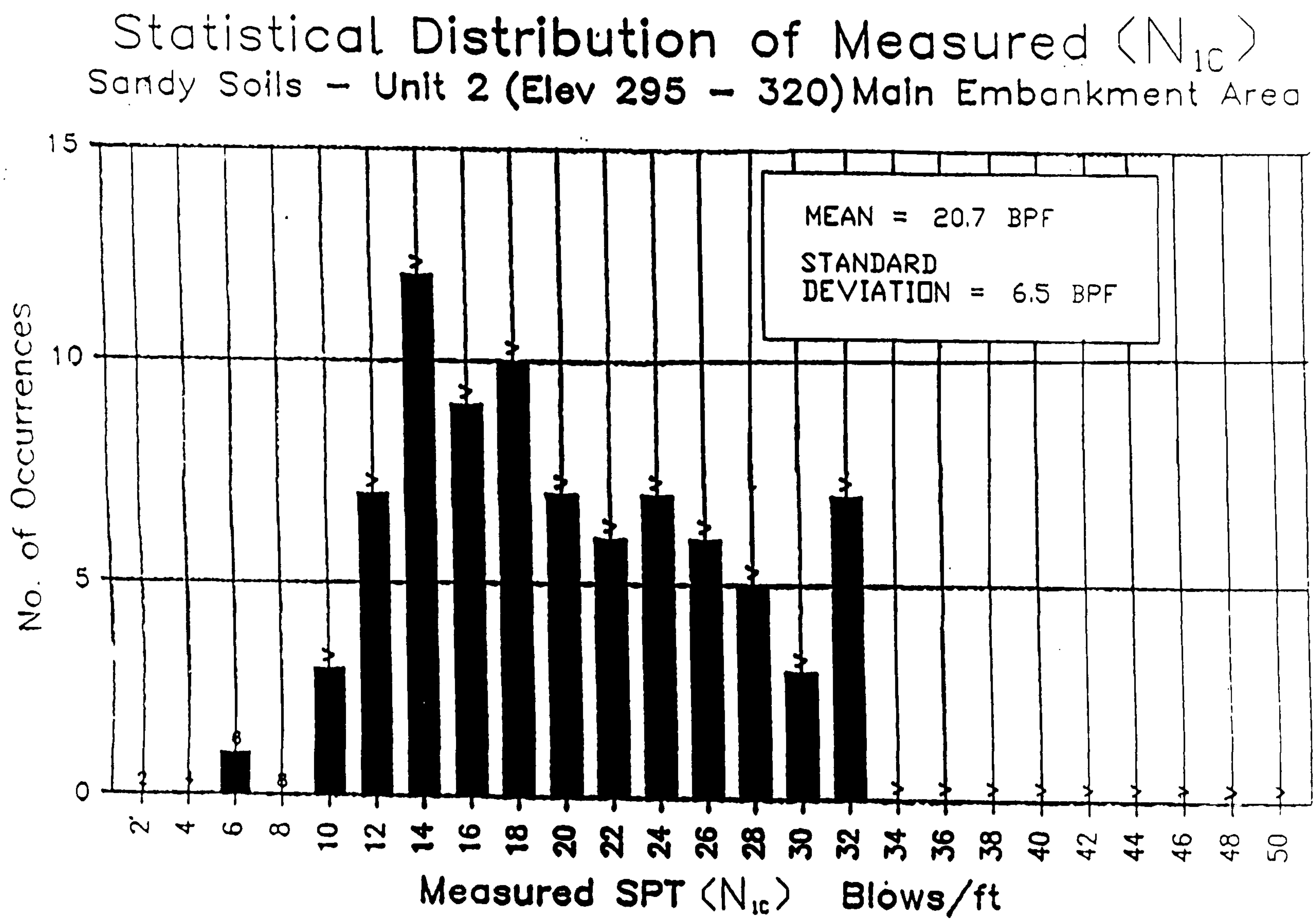


Figure 78. Statistical distribution of the equivalent clean sand blowcounts, Nlc, of the main embankment area of Unit 2 between elevations 305 and 320.

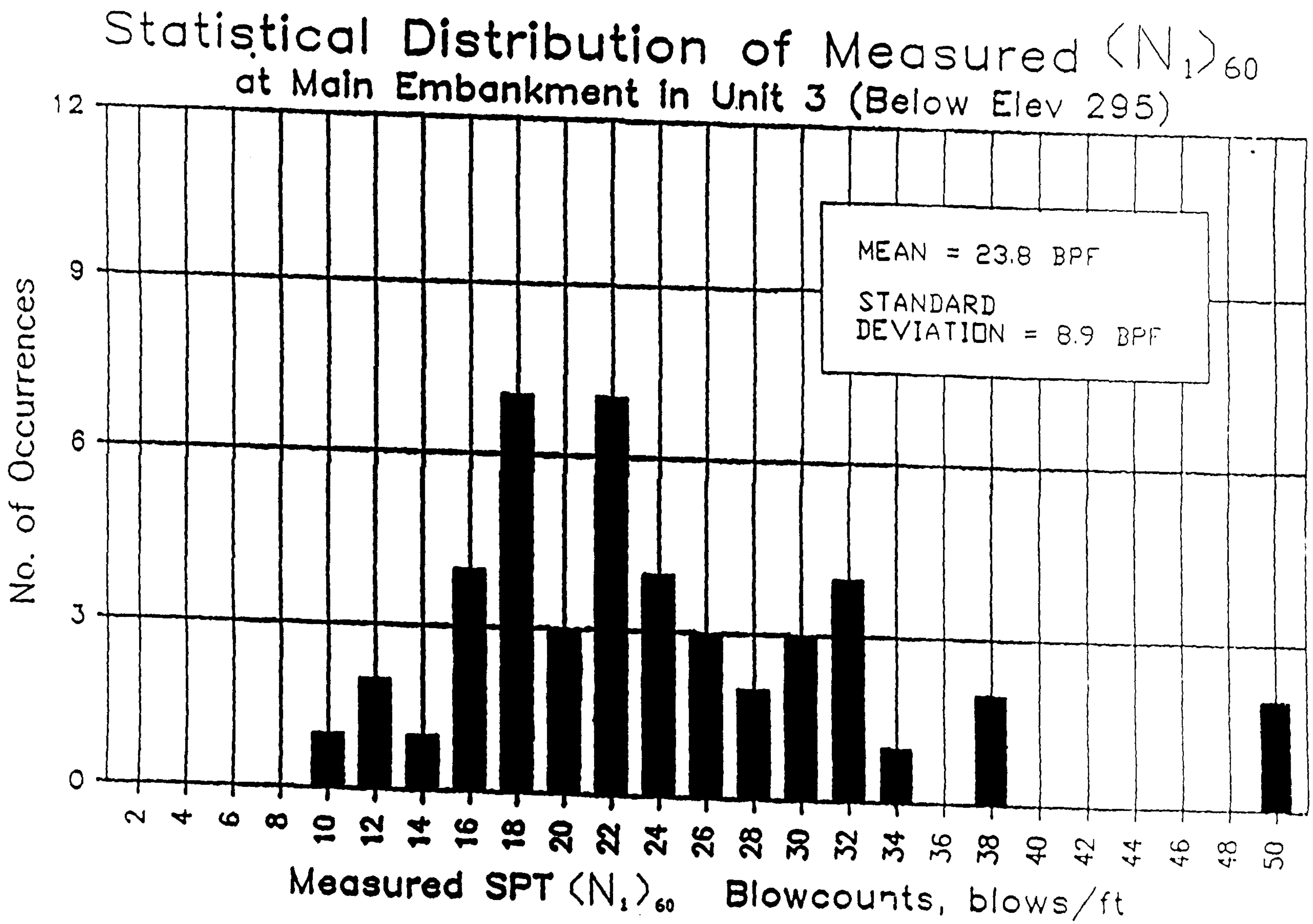


Figure 79. Statistical distribution of the overburden corrected blowcounts, $(N_1)_{60}$, of the main embankment area of Unit 3 below elevations 295.

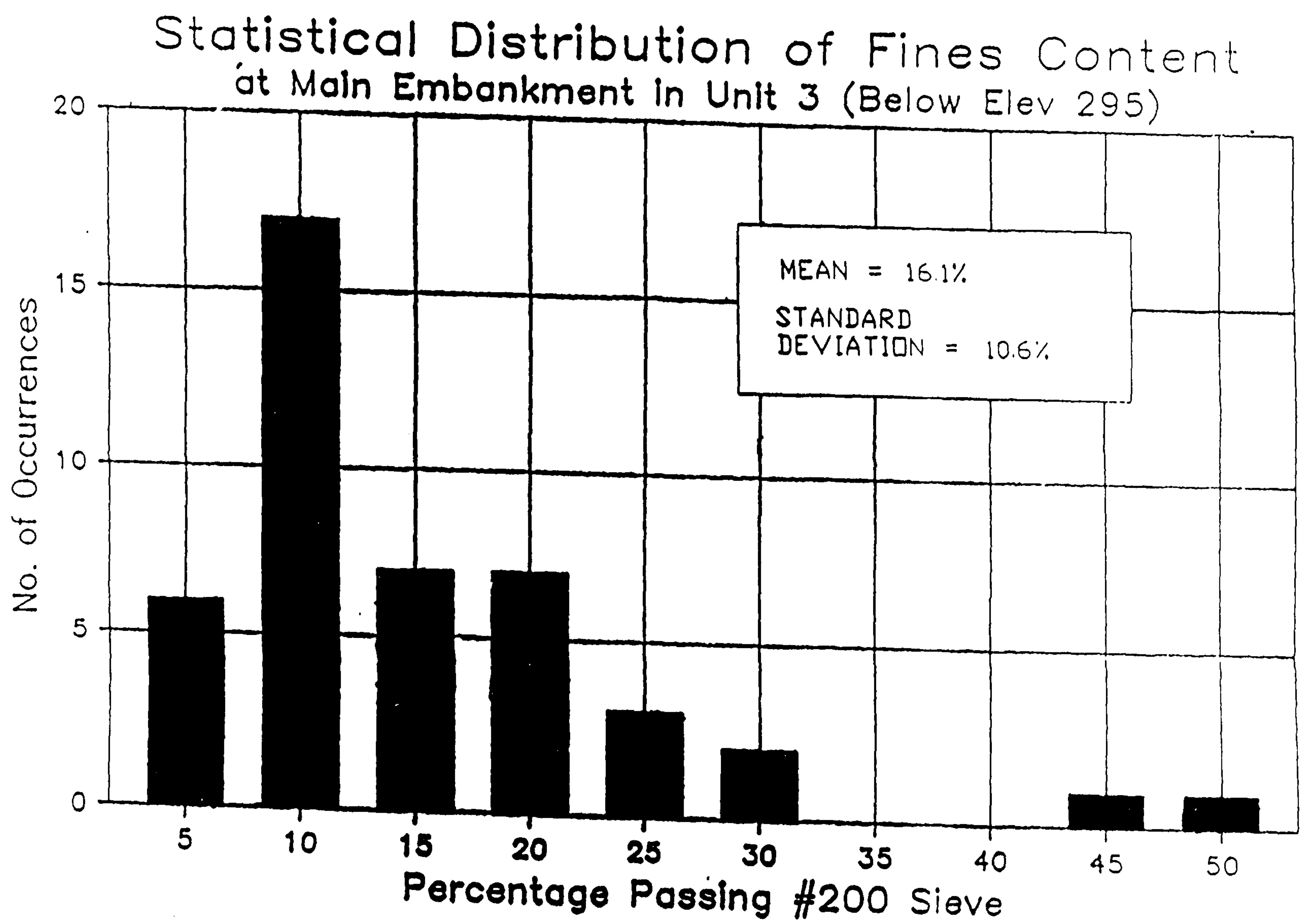


Figure 80. Statistical distribution of the fines content of the main embankment area of Unit 3 below elevations 295.

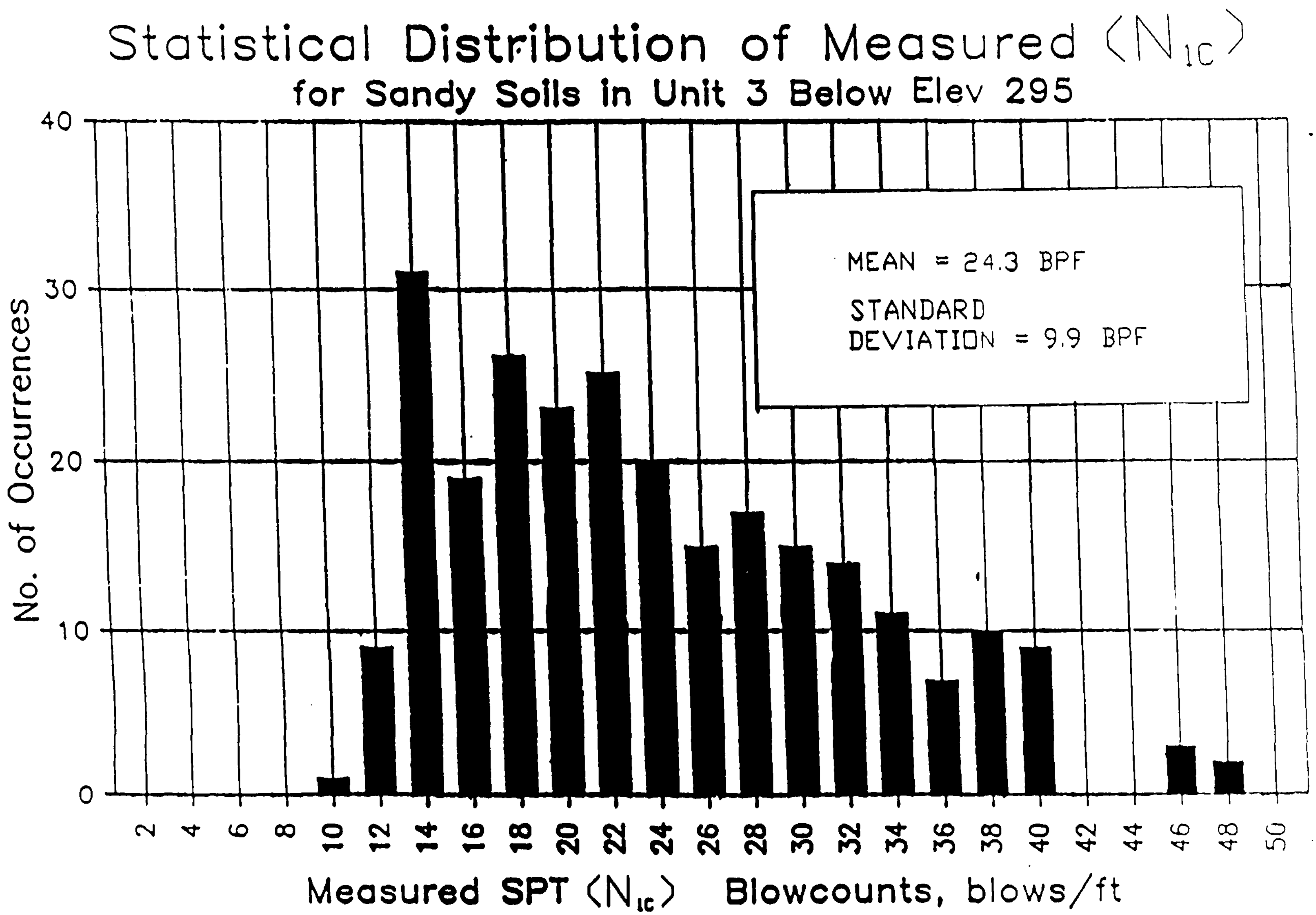


Figure 81. Statistical distribution of the equivalent clean sand blowcounts, N_{1c}, for both the main embankment and Switchyard areas of Unit 3 below elevations 295.

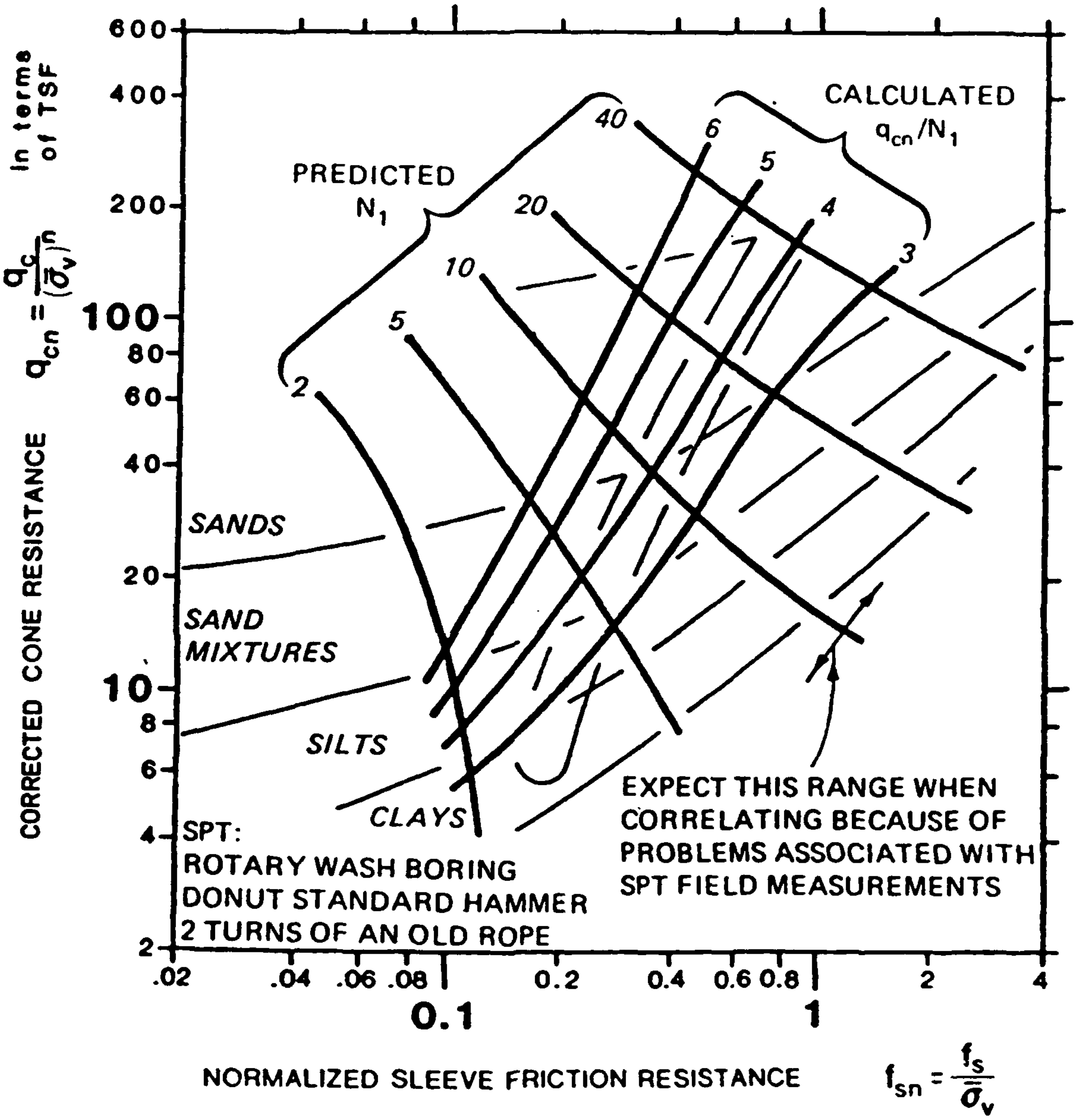


Figure 82. Chart used for CPT prediction of $(N_1)_{60}$.

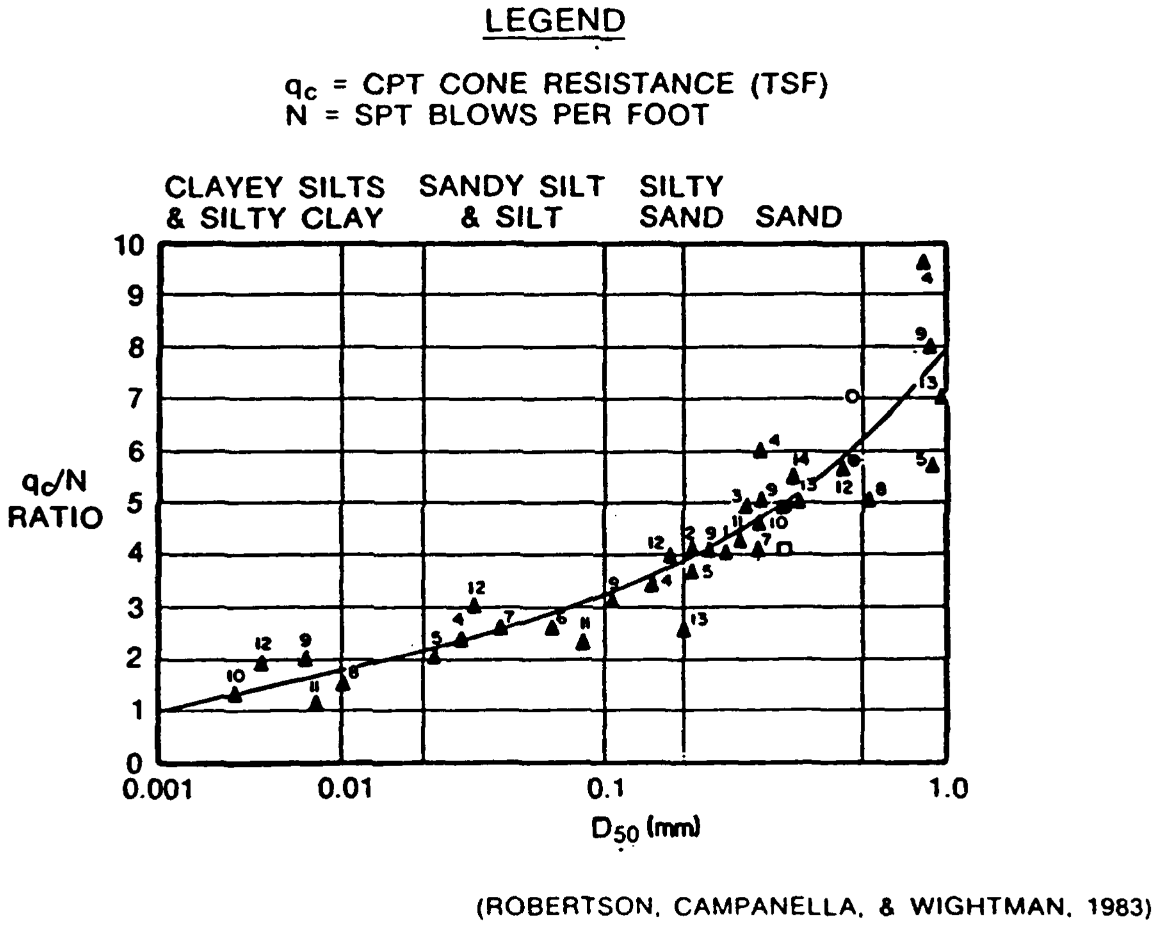


Figure 83. Correlation of D_{50} to q_c/N ratio.

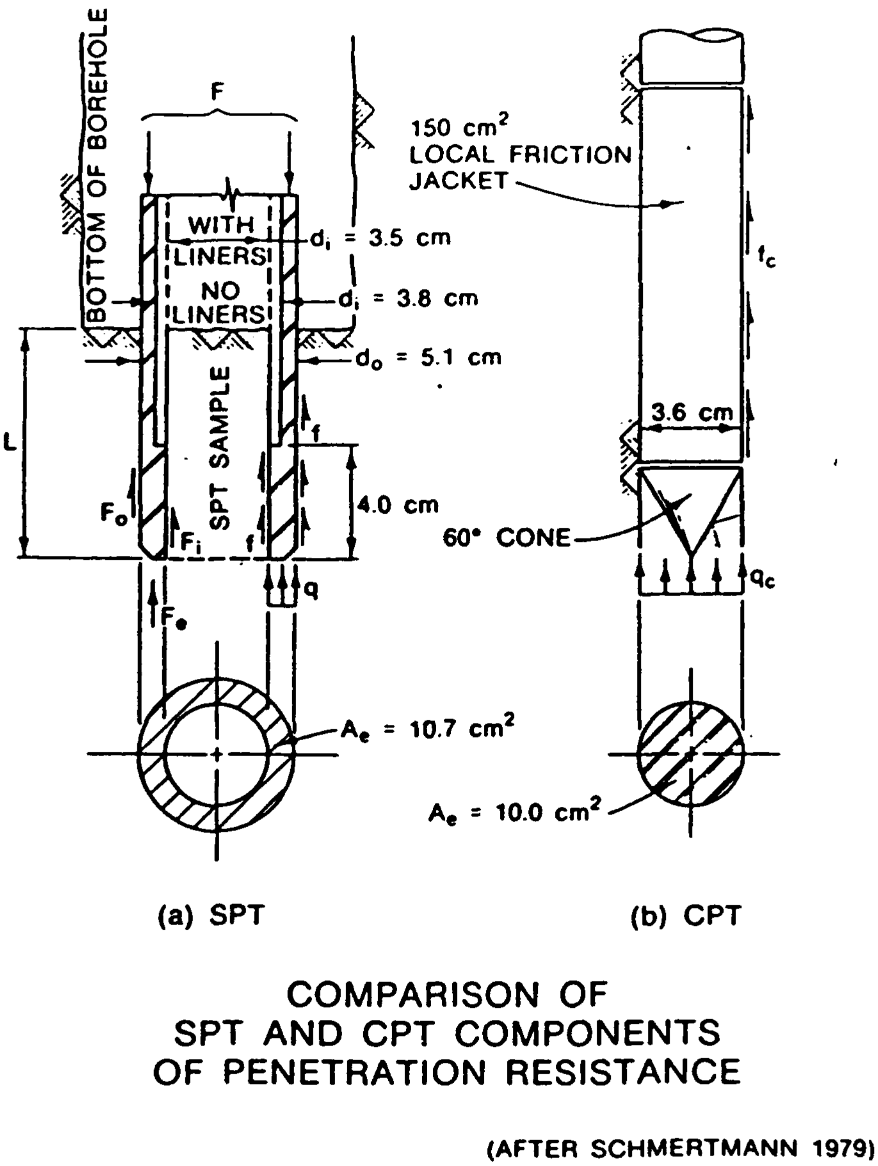


Figure 84. Comparison of SPT sampler end-bearing and side friction stresses to CPT probe measurements.

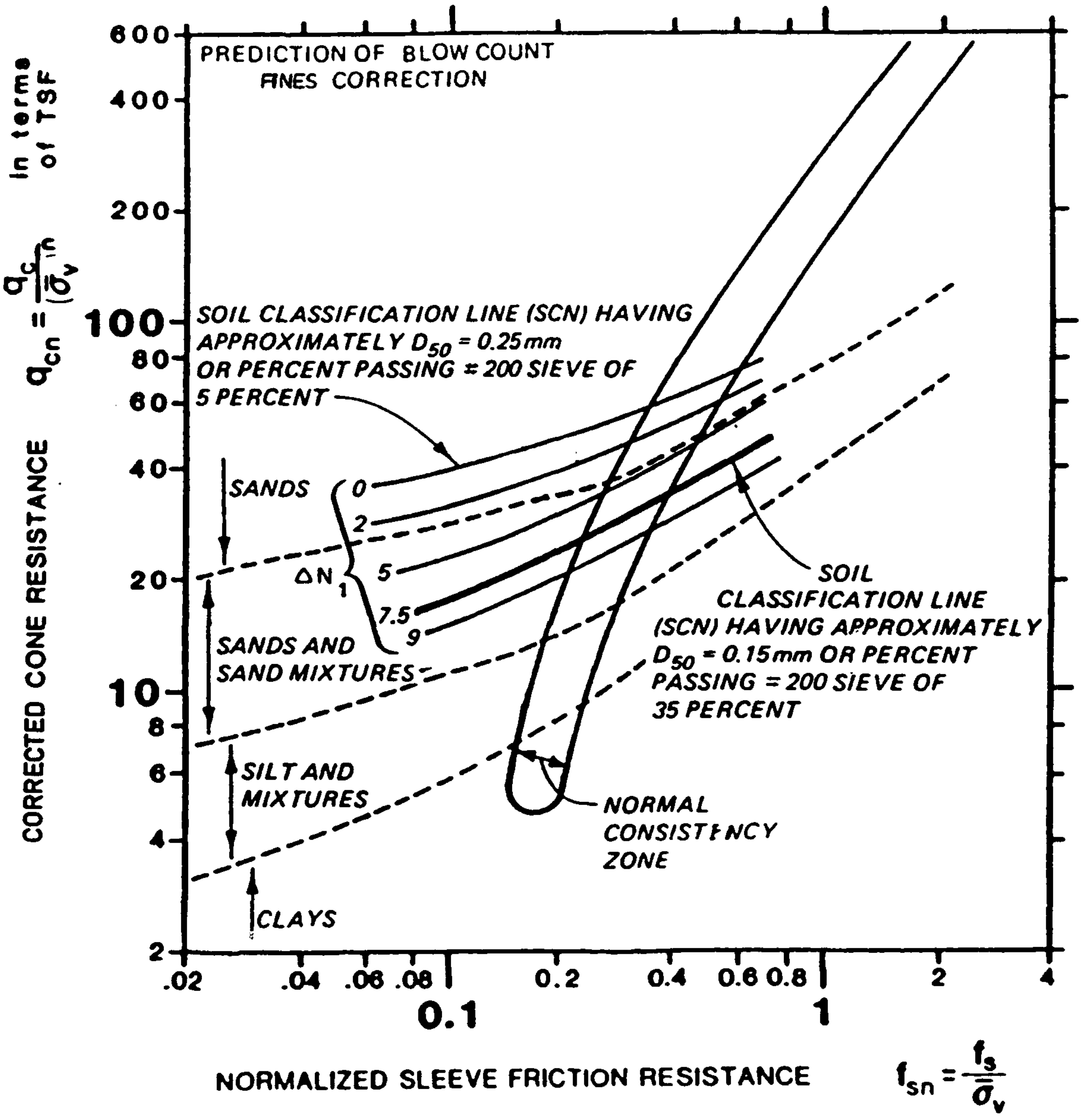


Figure 85. Chart used for CPT prediction of fines correction.

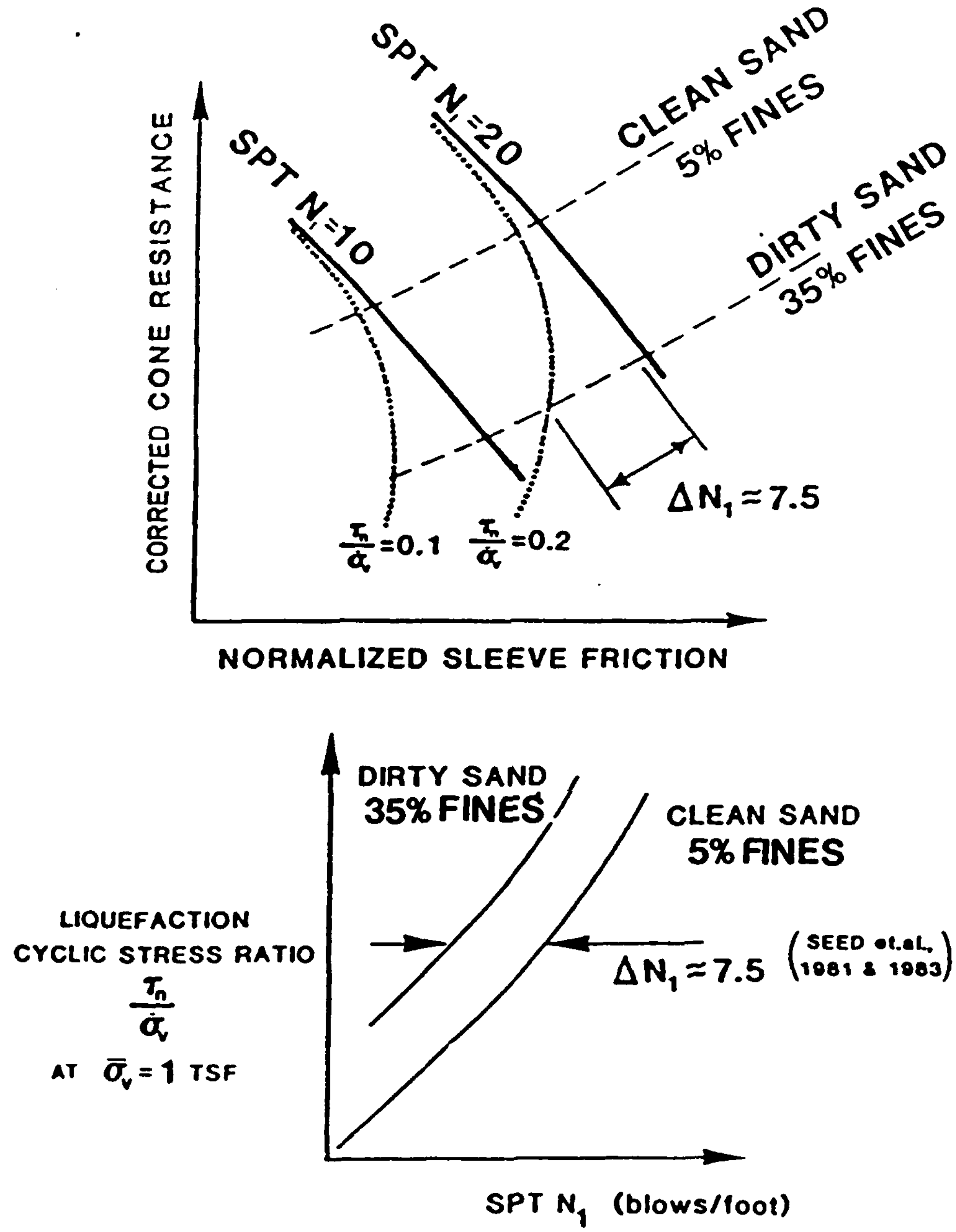


Figure 86. Illustration of SPT determination of $(N_1)_c$.

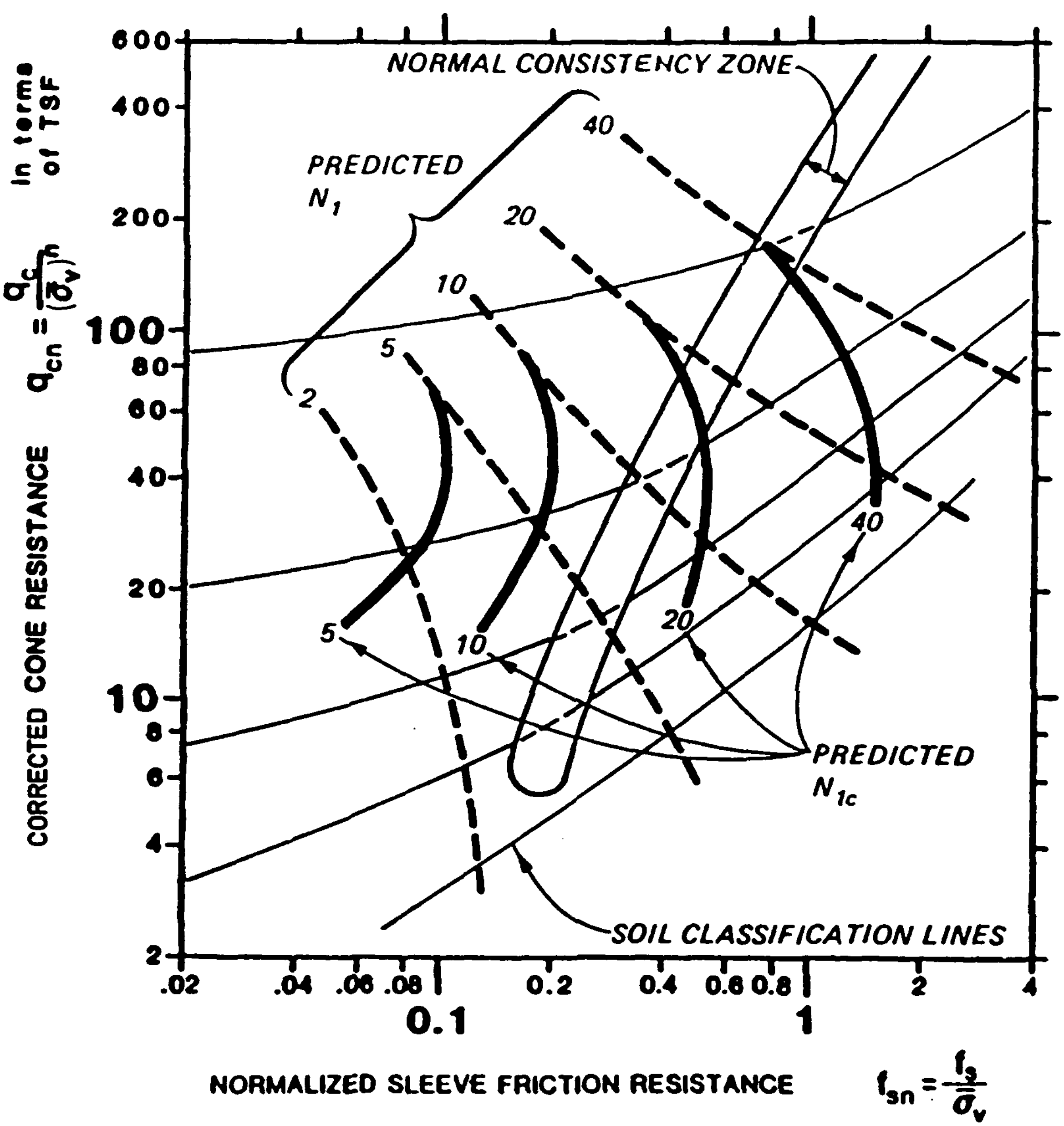


Figure 87. CPT prediction of silt corrected $(N_1)_c$ blowcount.

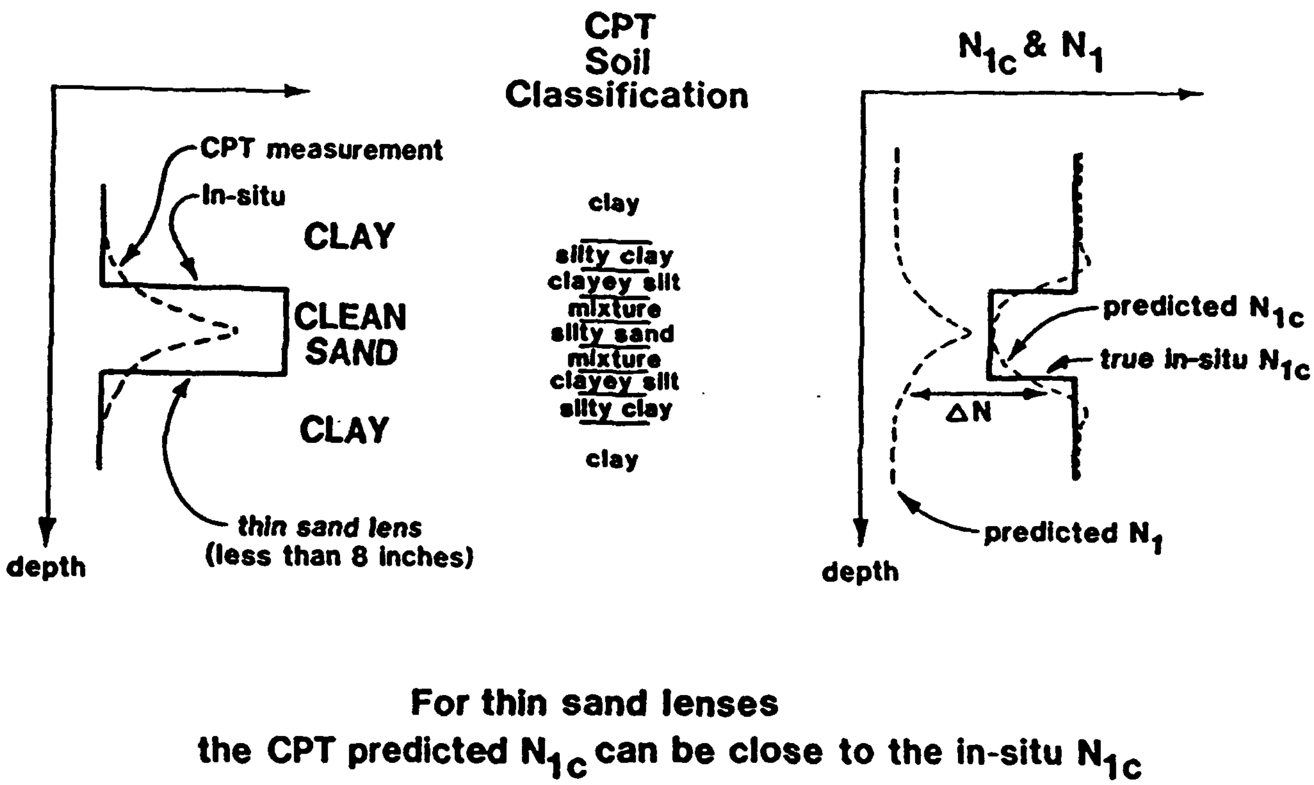


Figure 88. Effects of CPT probe penetration through soil layers on the prediction of N_1 and $(N_1)_c$.

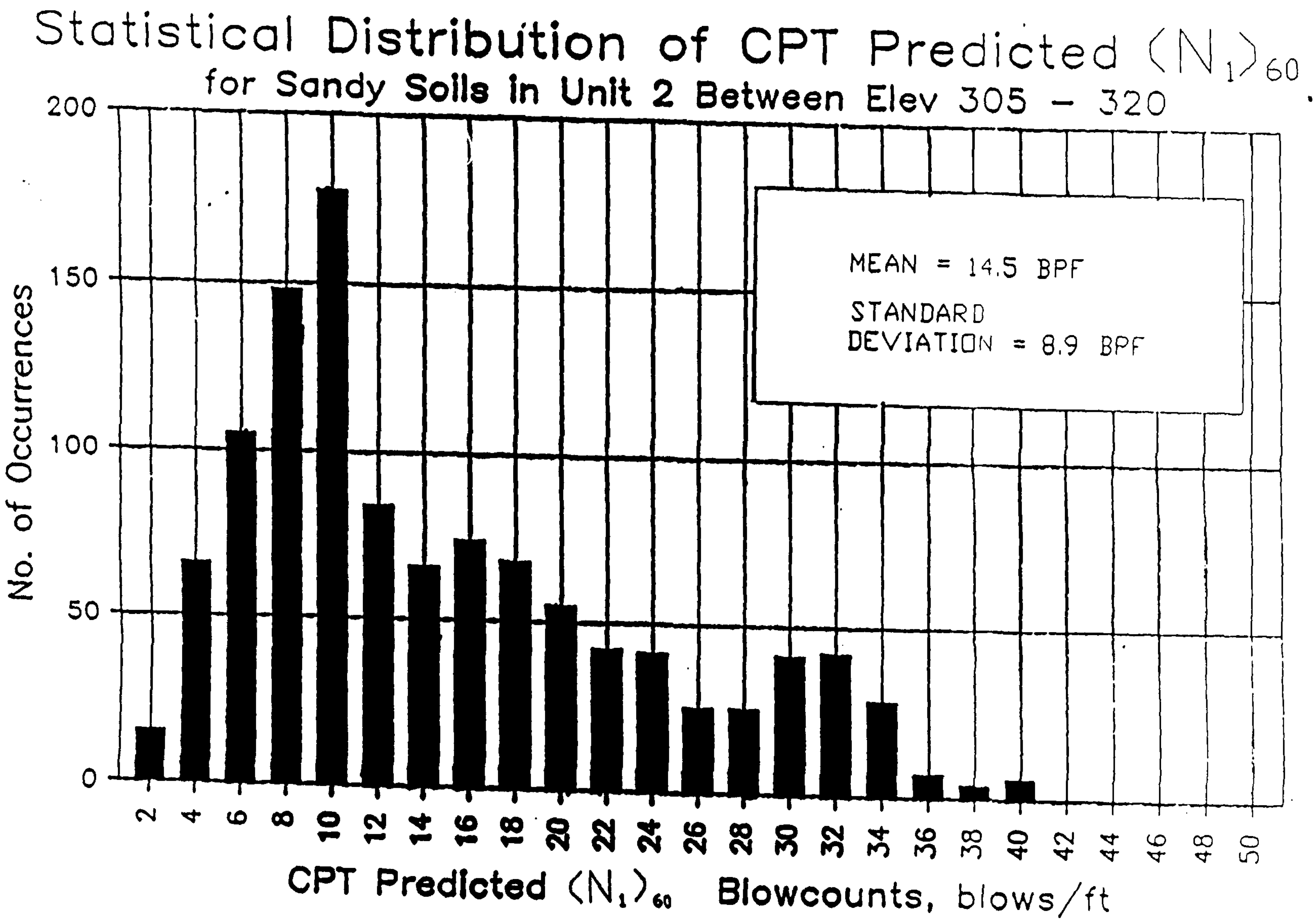


Figure 89. Statistical distribution of CPT predicted $(N_1)_{60}$ for sands in Unit 2 (elevation 305-320).

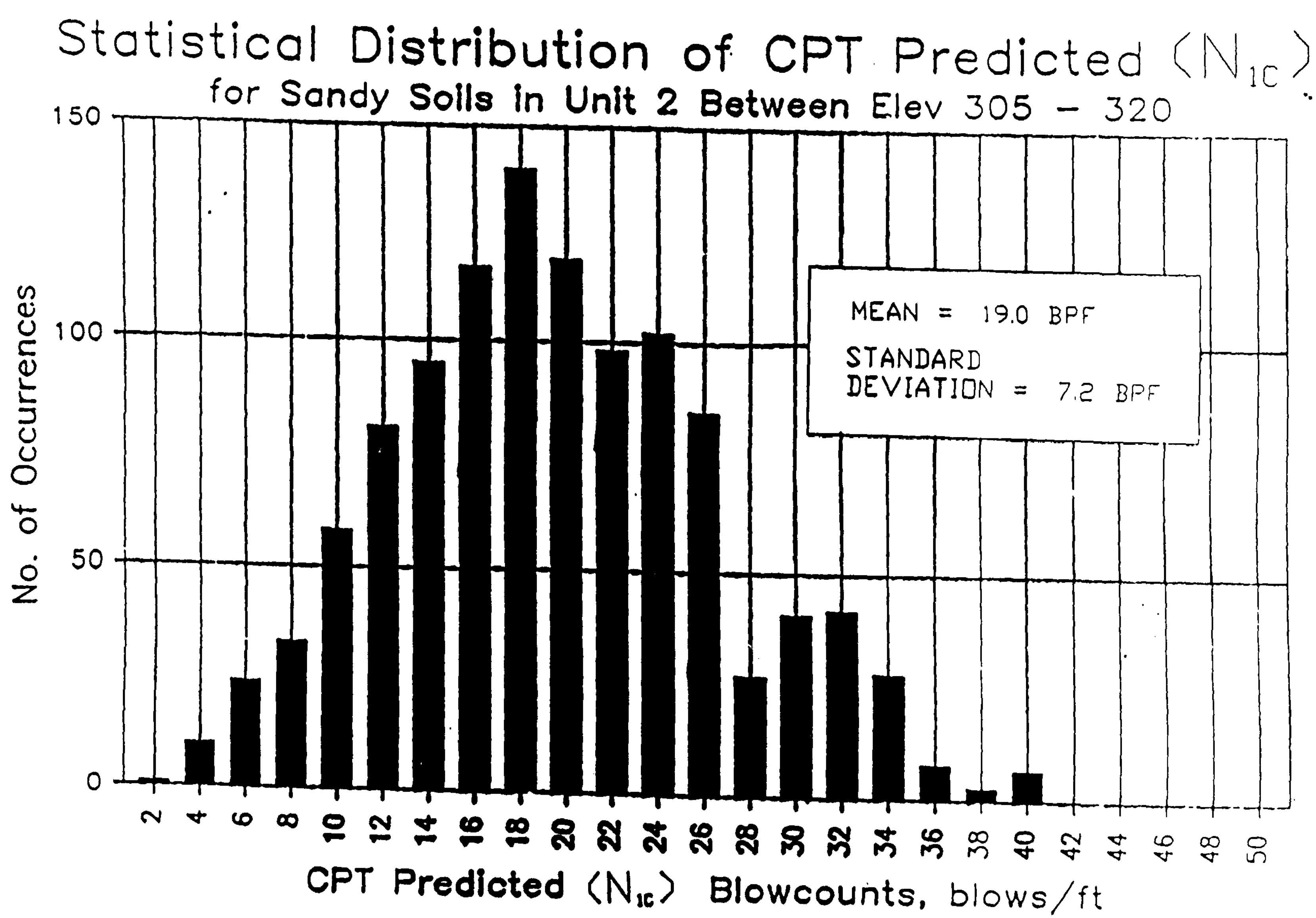


Figure 90. Statistical distribution of CPT predicted $(N_1)_c$ for sands in Unit 2 (elevation 305-320).

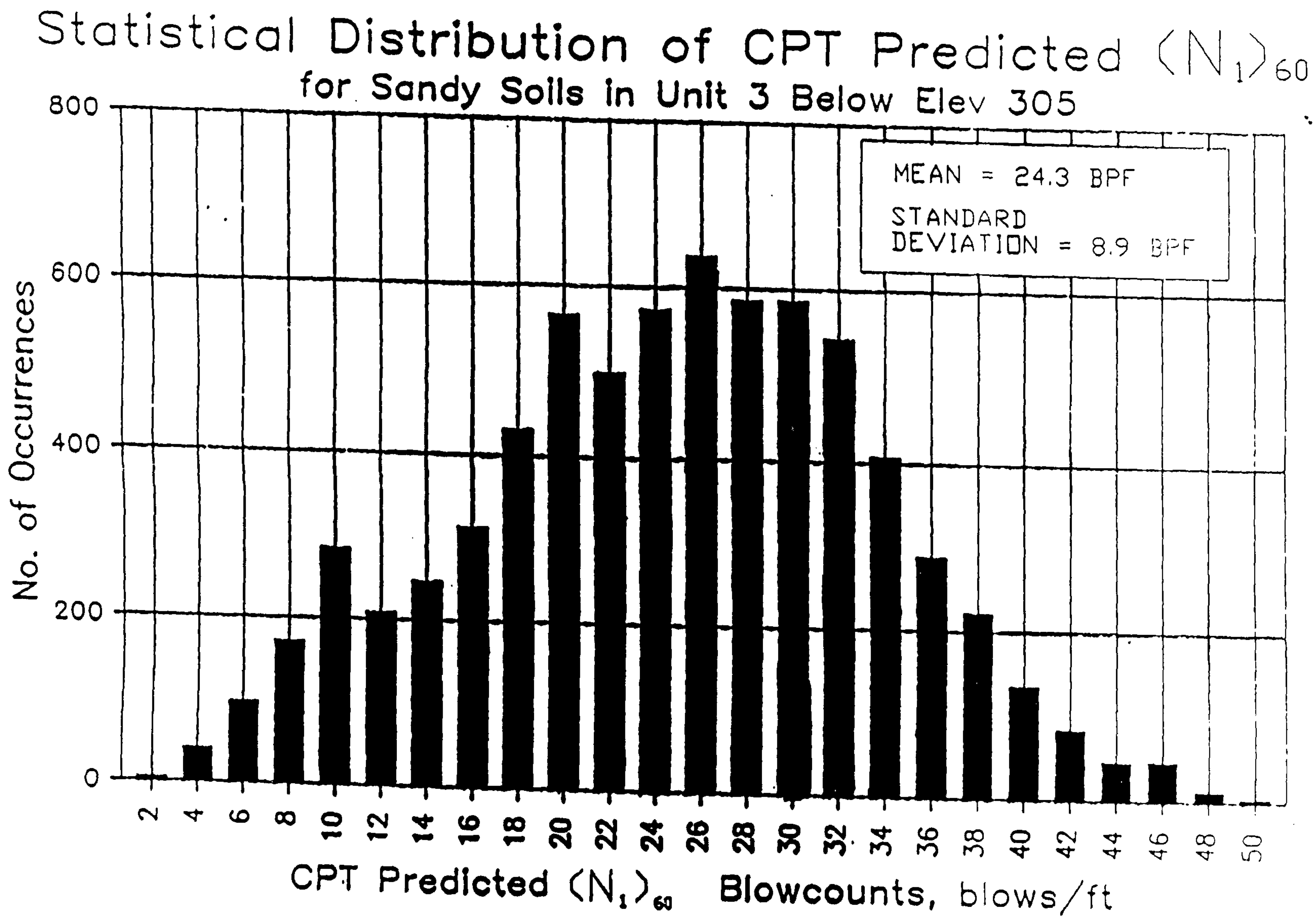


Figure 91. Statistical distribution of CPT predicted $(N_1)_{60}$ for sands in Unit 3 (below elevation 305).

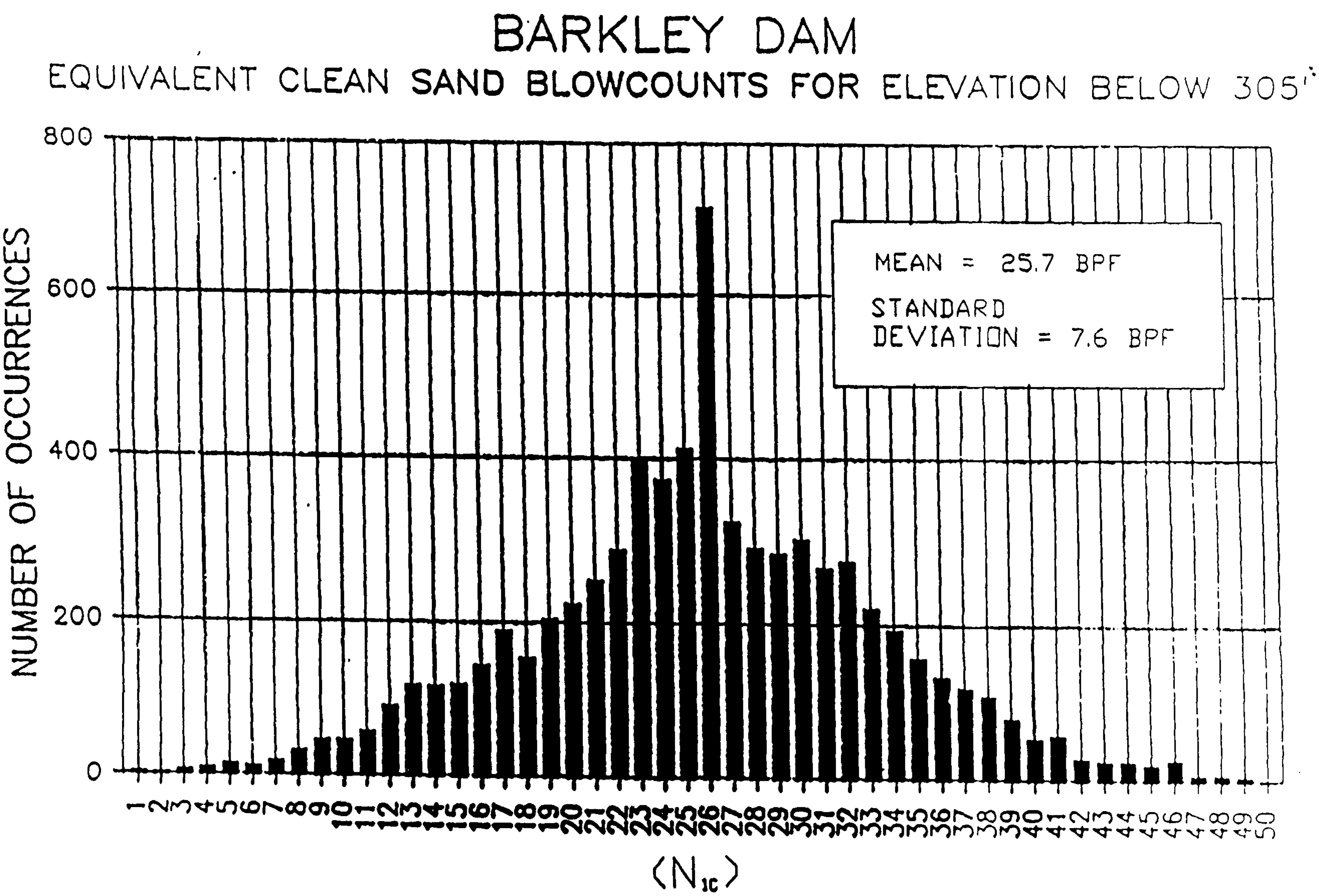


Figure 92. Statistical distribution of CPT predicted $(N_1)_c$ for sands in Unit 3.

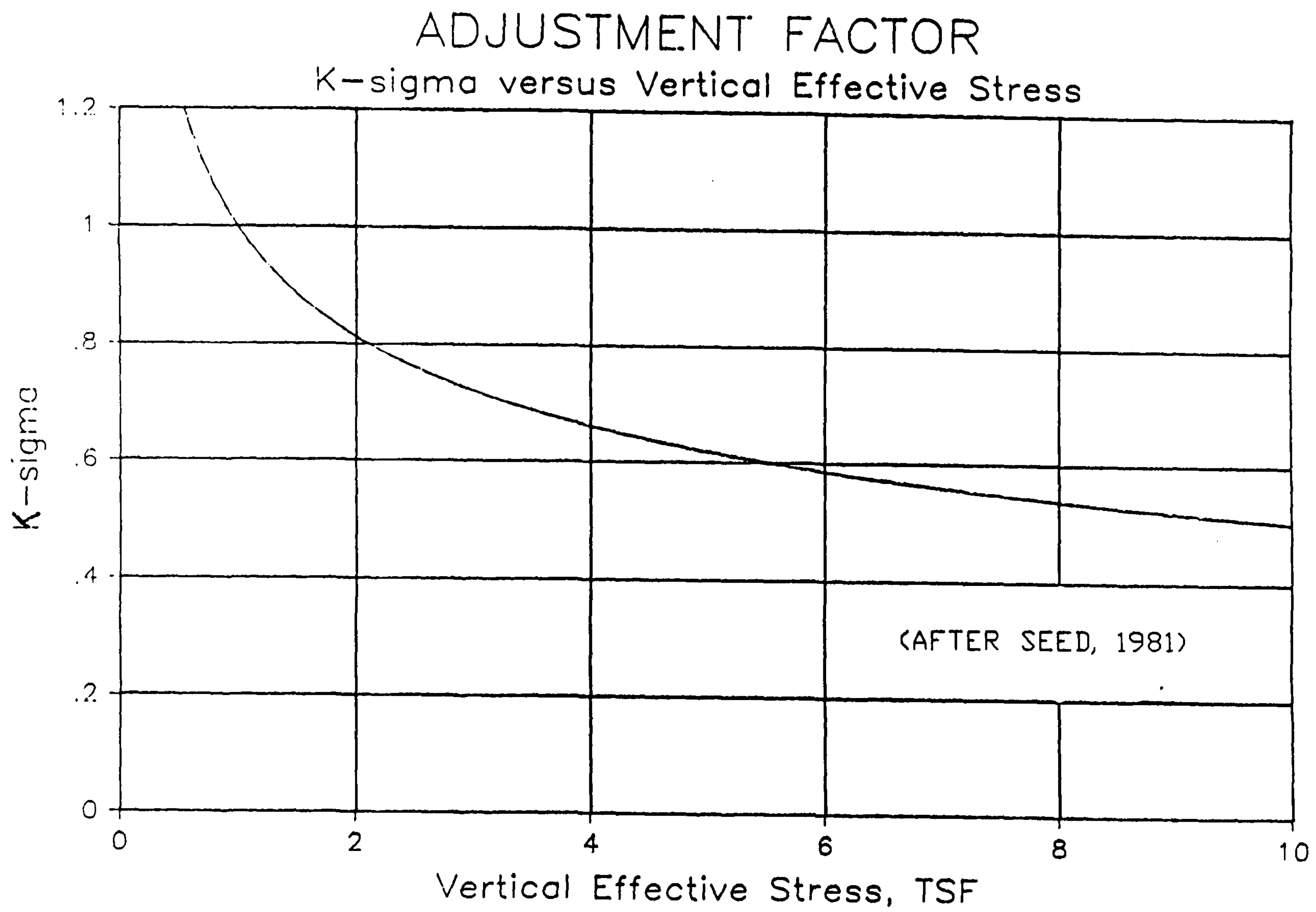


Figure 93. K_{sigma} adjustment factor.

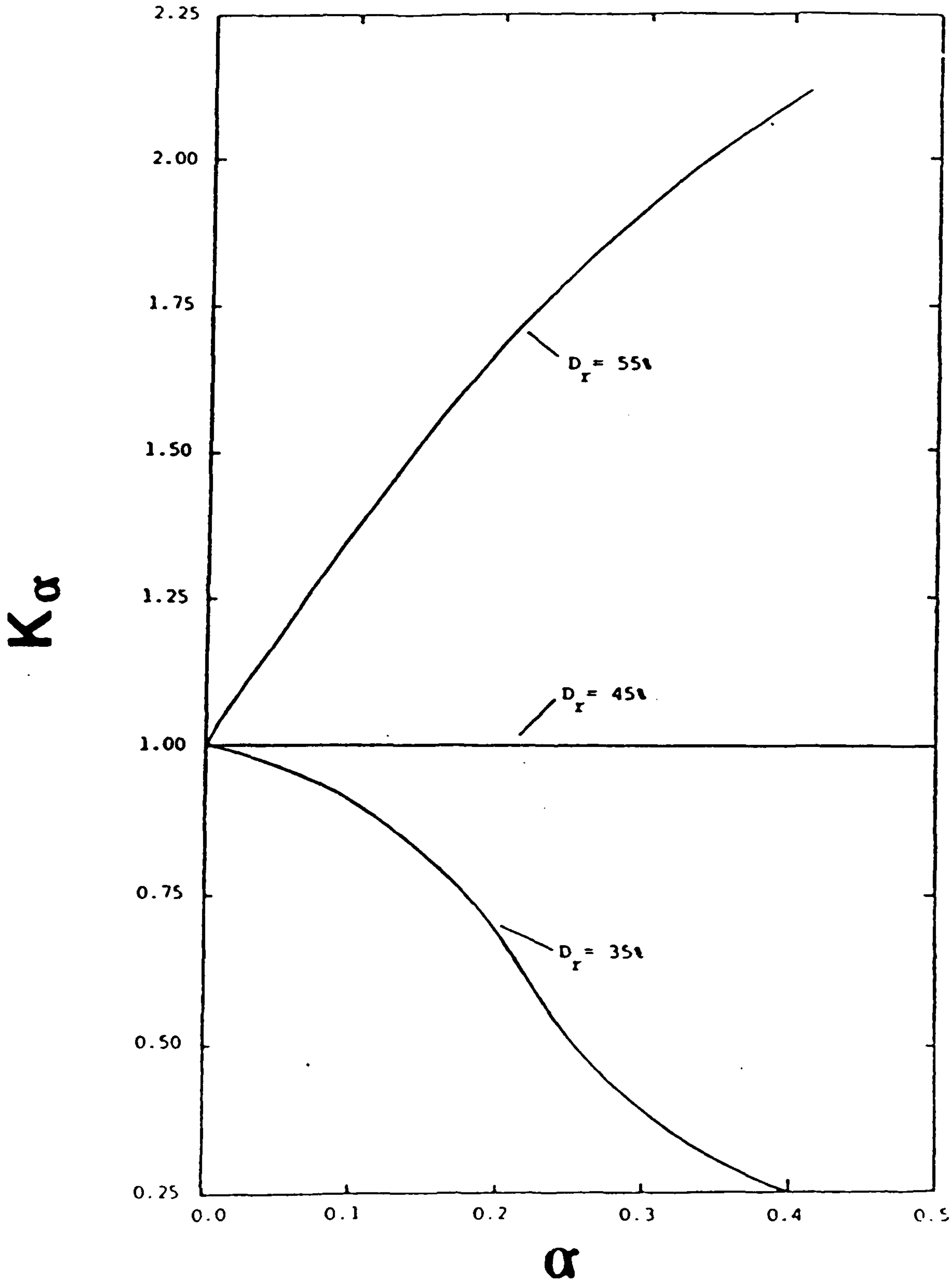


Figure 94. K_{alpha} adjustment factor.

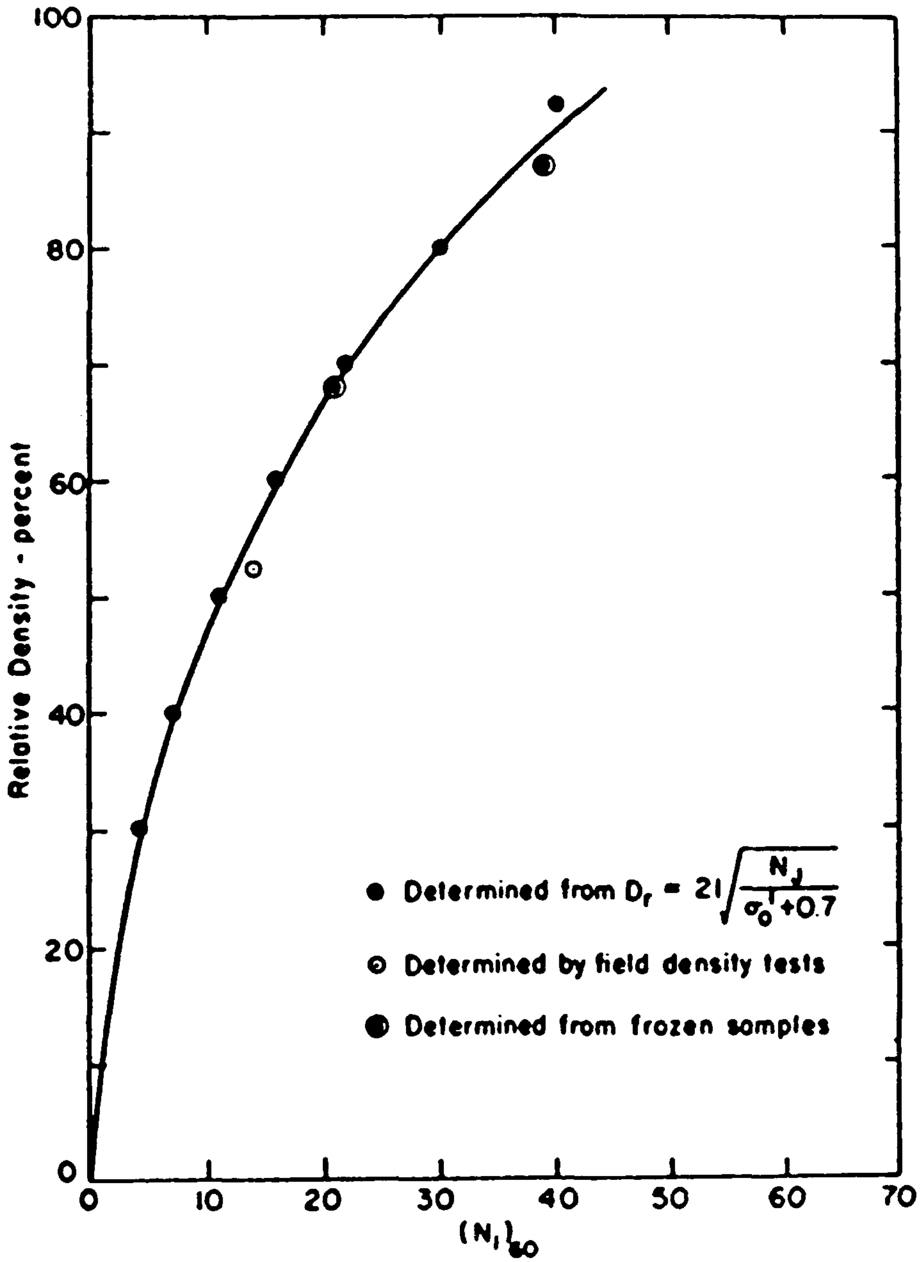


Figure 95. Relationship between relative density and $(N_1)_{60}$ (Tokimatsu and Seed, 1987).

Note: Unit 3b modeled
Unit 3b is desig

Cyclic Strengths Based on CPT Predicted $(N_1)60$ Blowcounts (Mean — $1/2\sigma$) for Sandy Soils

	$(N_1)_c$ — 30%ile	$\left(\tau/\sigma'_v\right)_{\sigma=0,\ \sigma=1\ tsf,\ M=8^+}$
Unit 2	15 bpf	0.184
Unit 3a & 3c	22 bpf	0.214

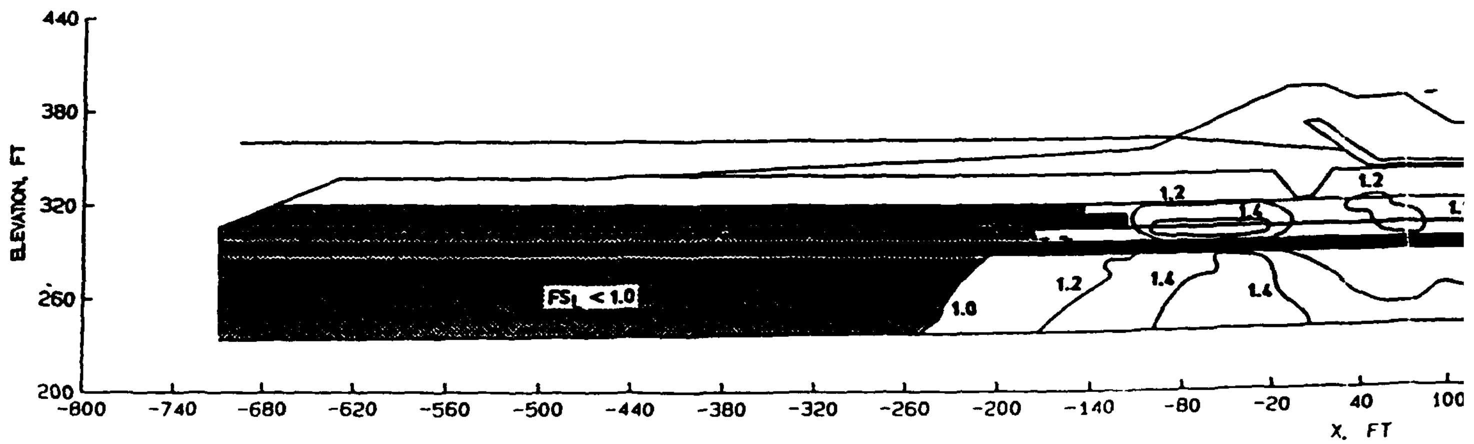
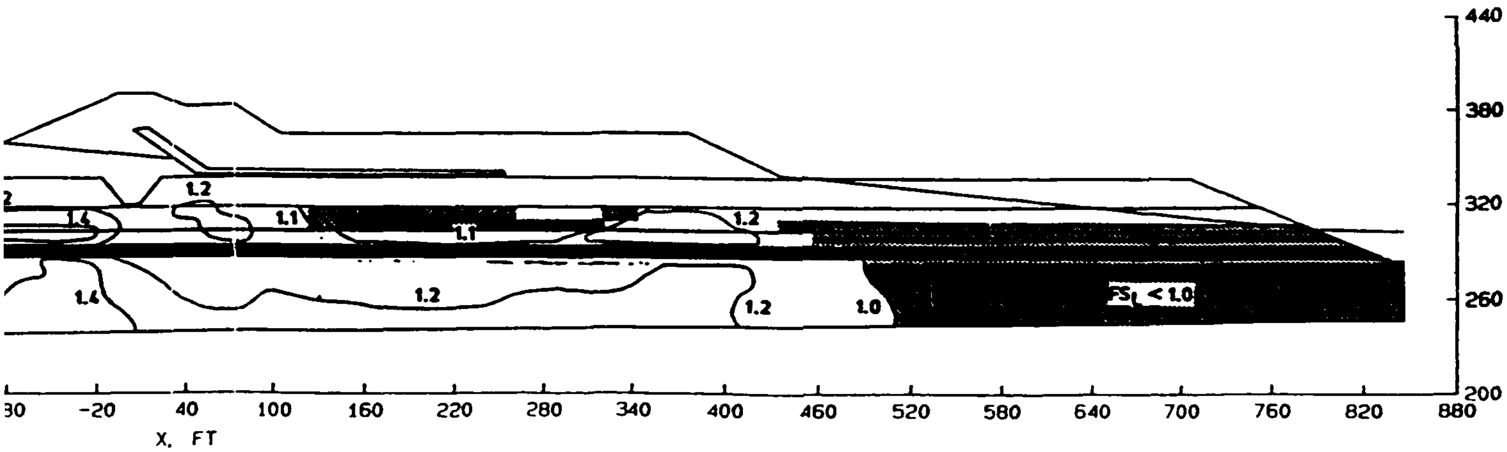


Figure 96. Switchyard Area: Contours
against liquefaction in th

Note: Unit 3b modeled as non—liquefiable.
Unit 3b is designated by ▬.



:d Area: Contours of the safety factor
Liquefaction in the foundation sands.

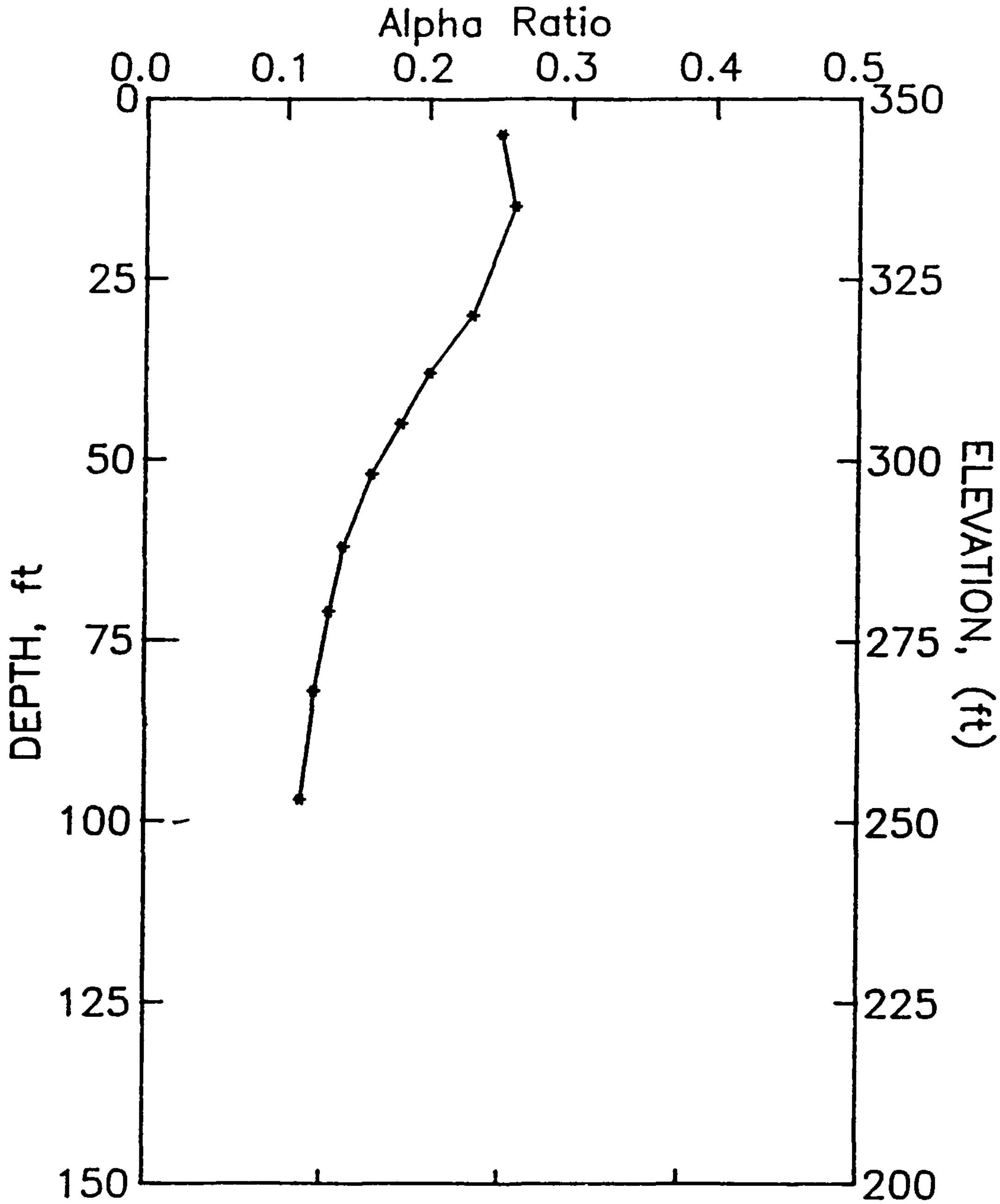


Figure 97. Main Embankment Area: Alpha values from static analysis for Profile 1.

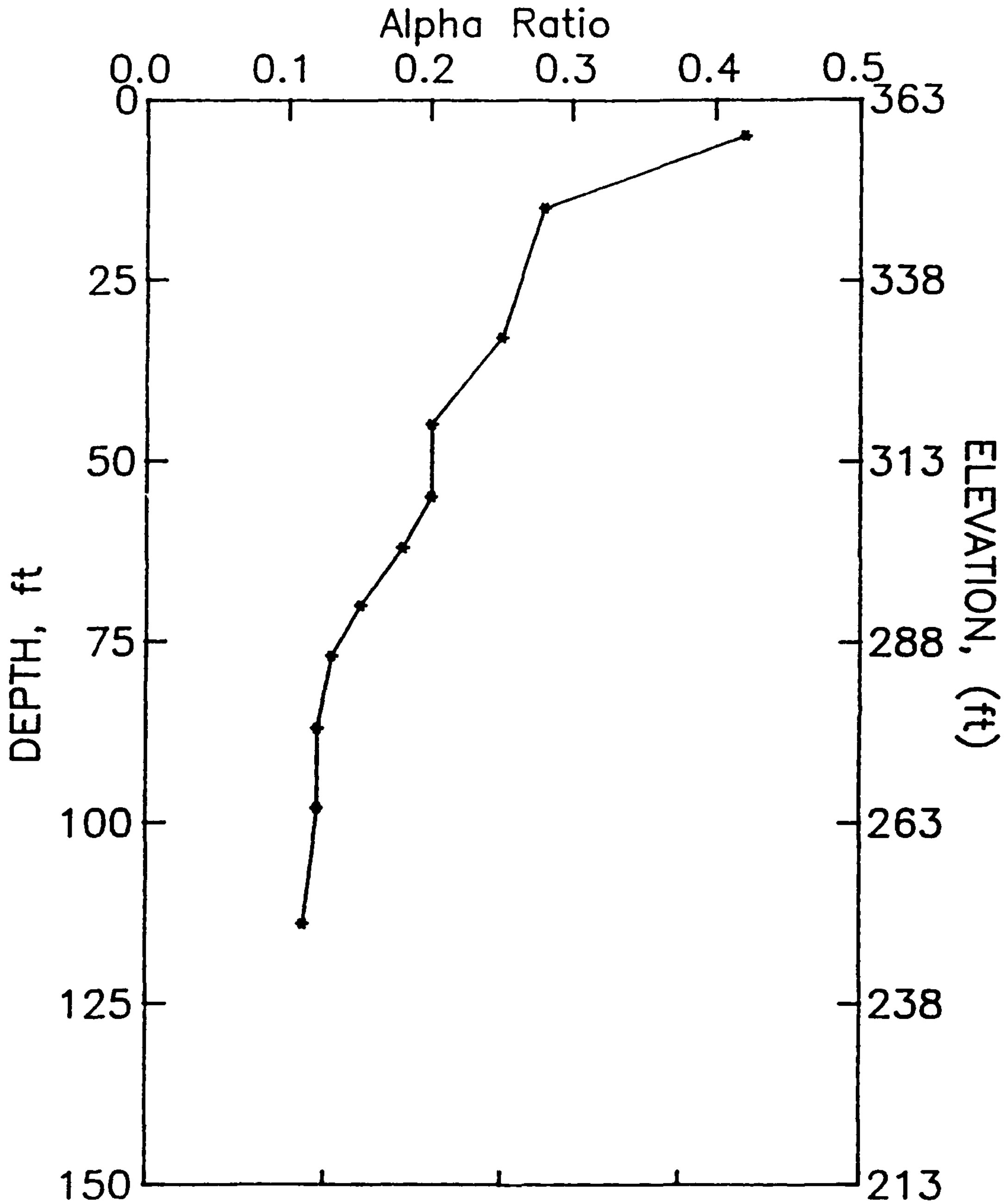


Figure 98. Main Embankment Area: Alpha values from static analysis for Profile 2.

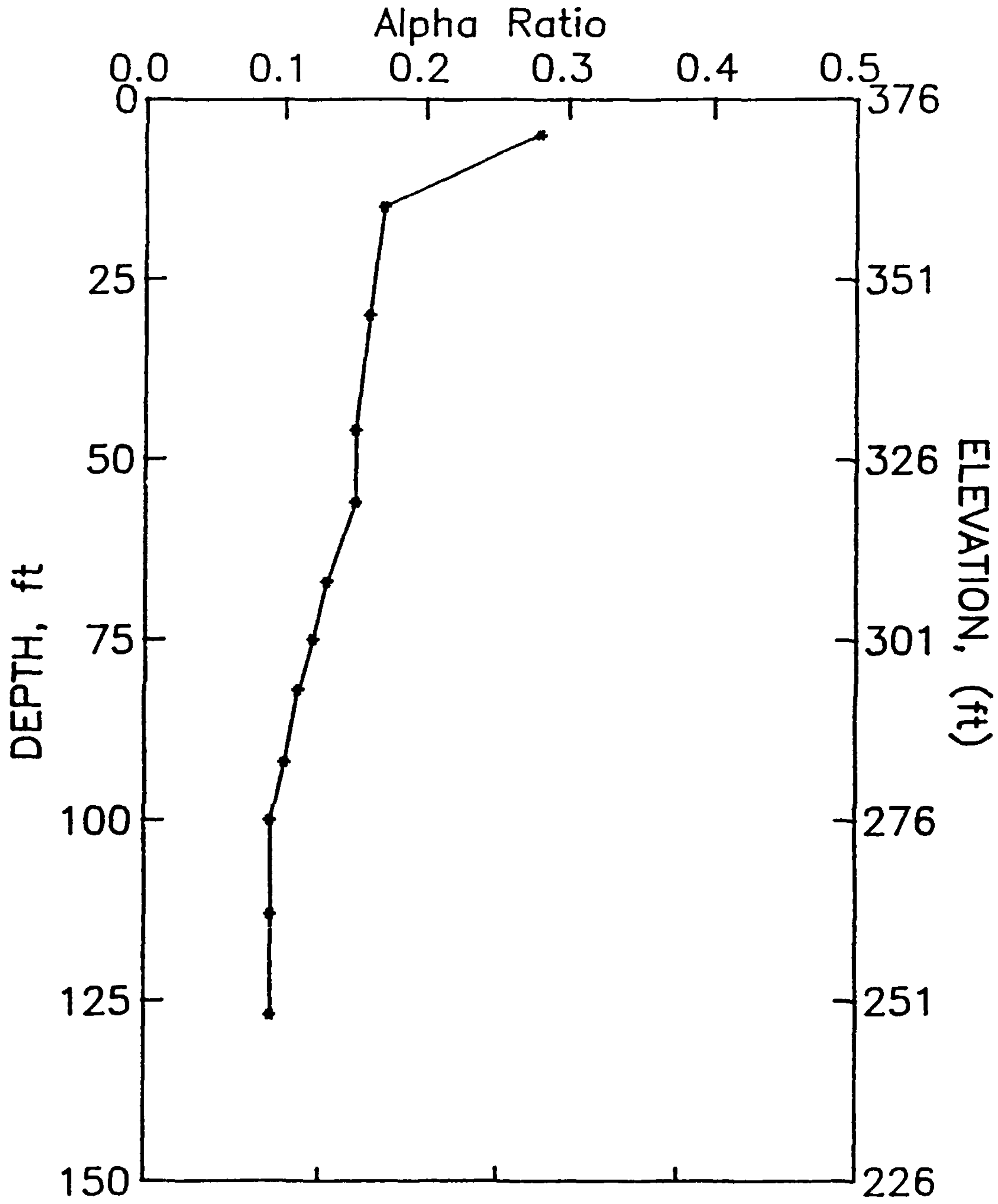


Figure 99. Main Embankment Area: Alpha values from static analysis for Profile 3.

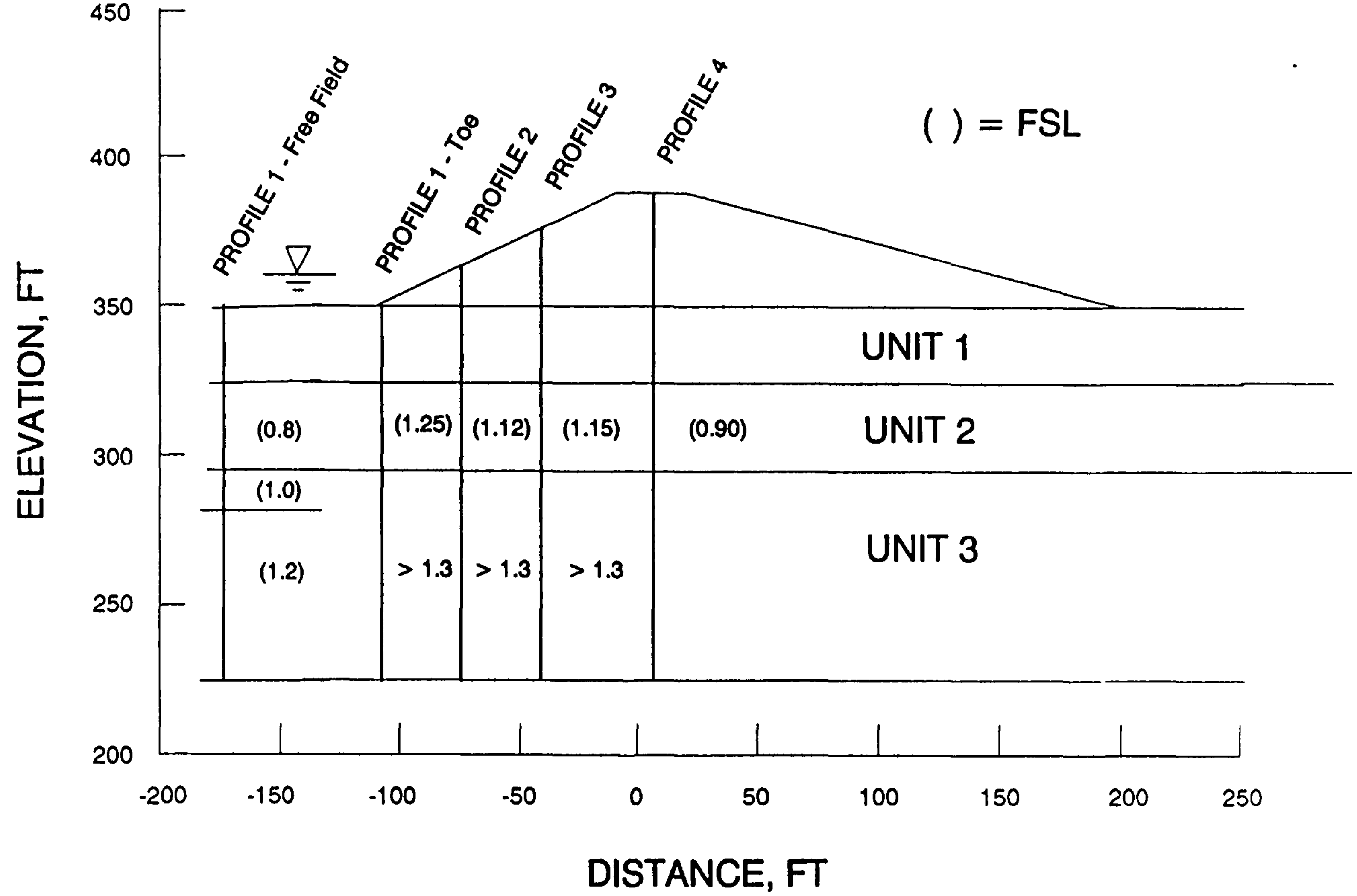


Figure 100. Main Embankment Area: Cross section showing safety factors against liquefaction.

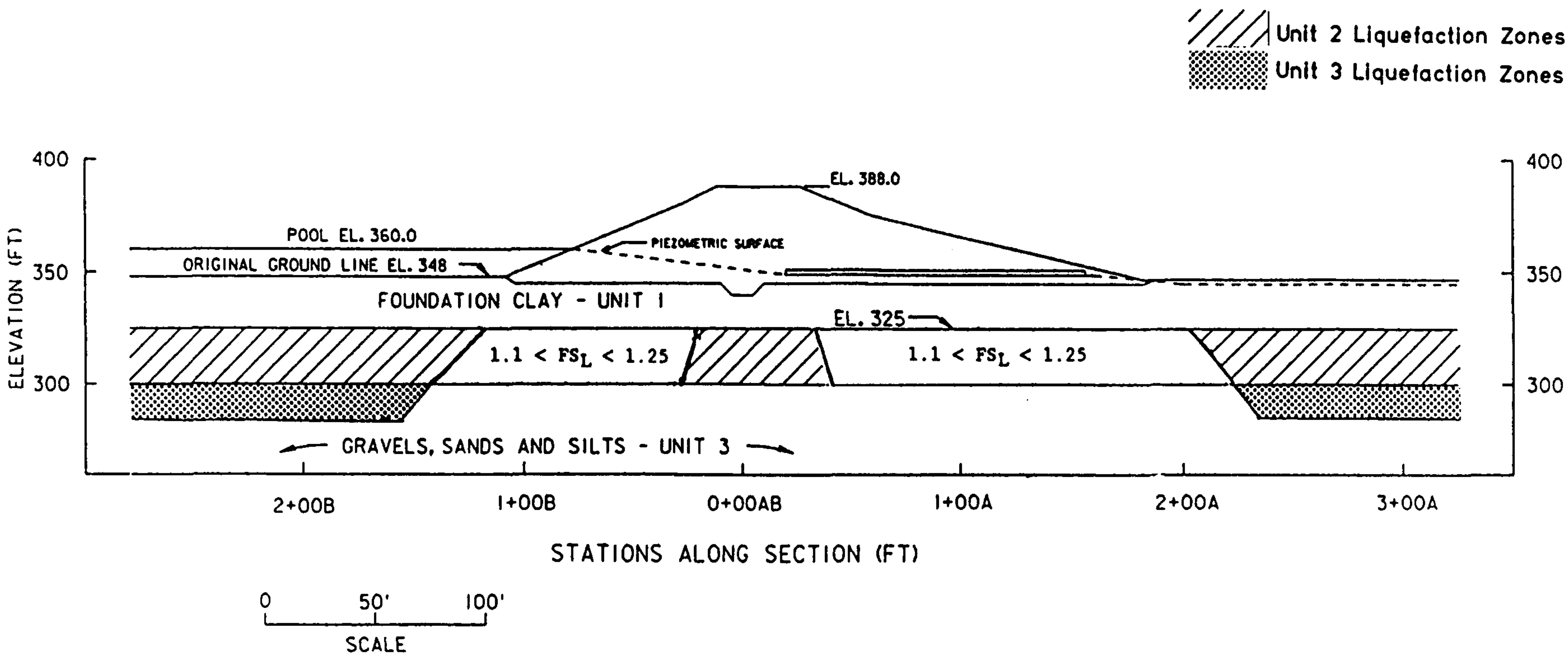


Figure 101. Interpreted zones of liquefaction in the foundation beneath the main embankment.

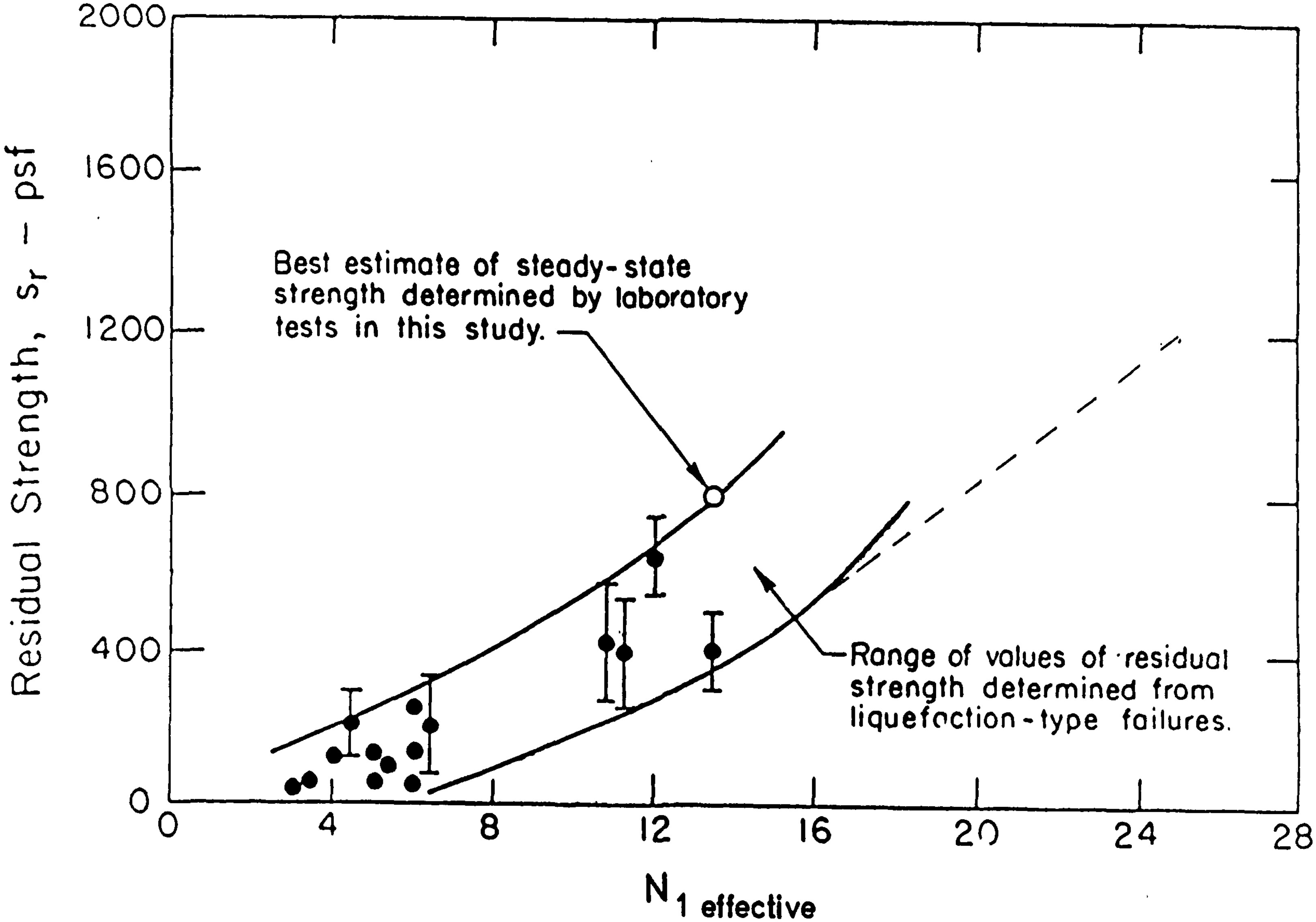


Figure 102. Comparison between values if residual strength and steady-state strength determined in this study.

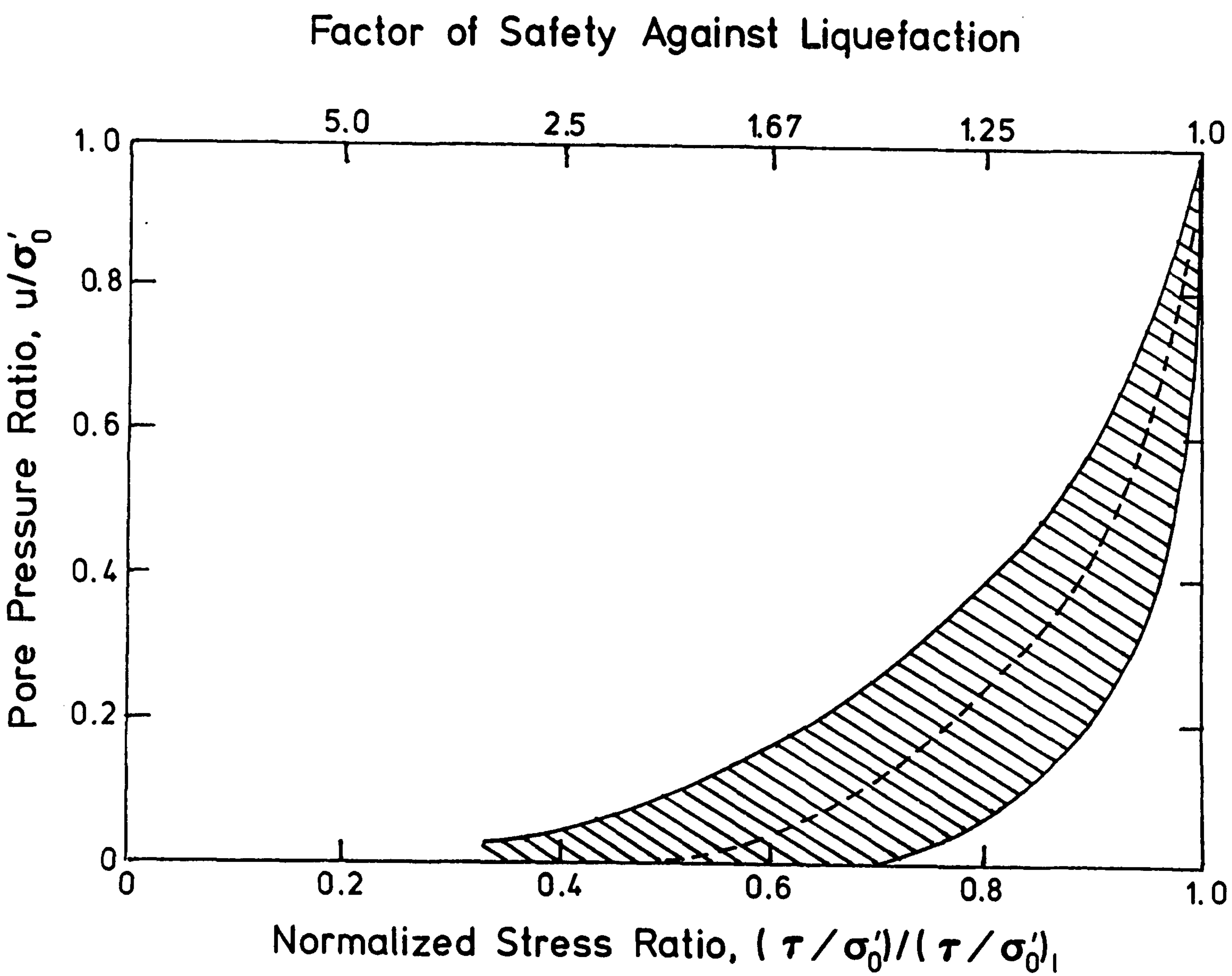


Figure 103. Normalized pore pressure ratio.

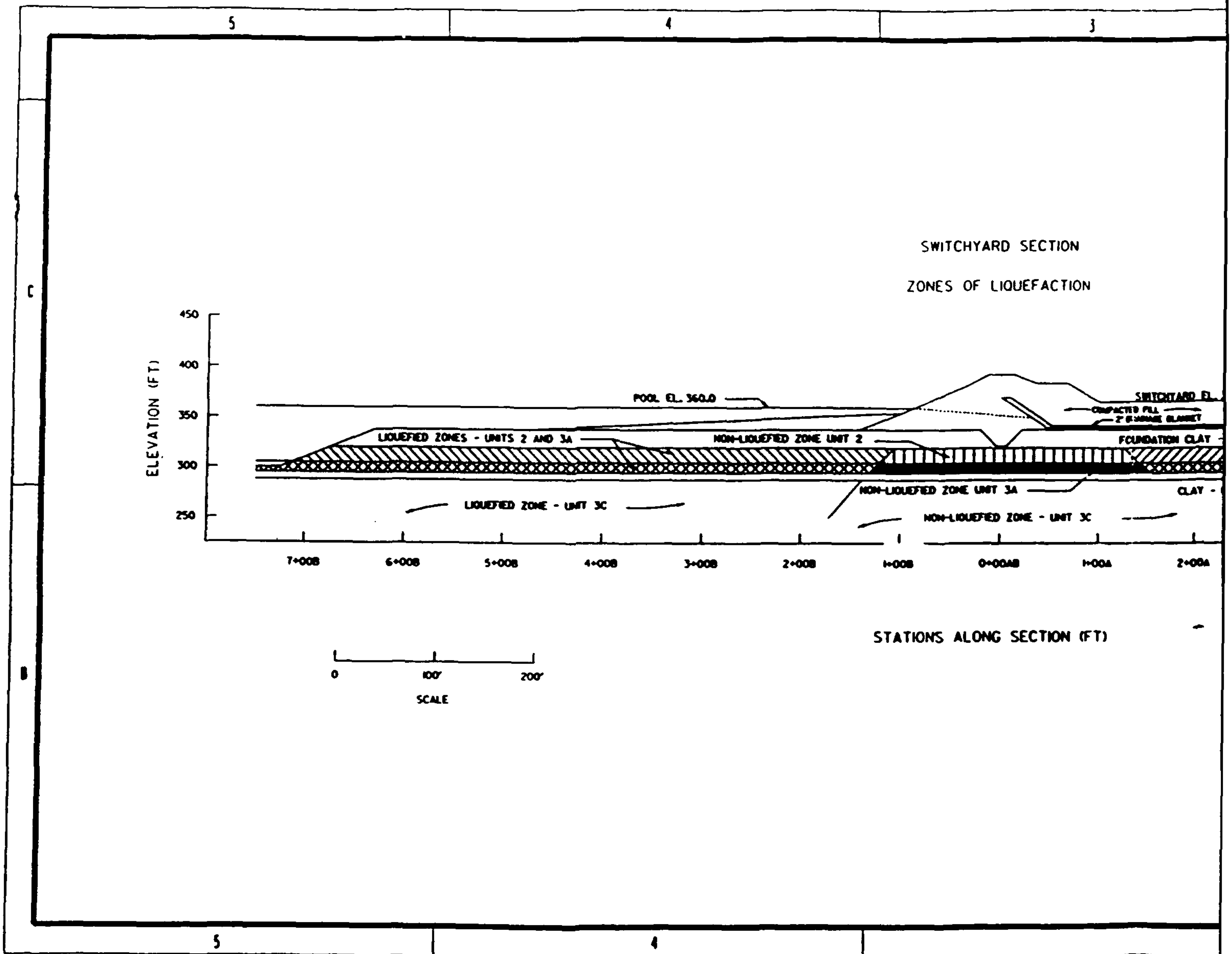


Figure 104. Post-earthquake strengths for

3 | 2 | 1

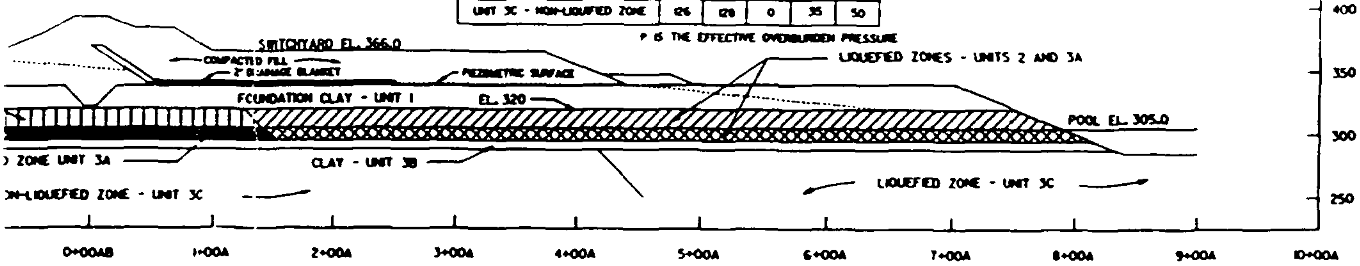
STABILITY ANALYSIS PARAMETERS

MATERIAL	UNIT WEIGHT		c (PSF)	ϕ	% Ru
	γ_m	γ_{sat}			
COMPACTED FILL	126	128	800	18	----
RANDOM FILL	126	128	320	6	----
UNIT 1 CLAY	115	125	960	12	----
UNIT 2 - LIQUEFIED ZONE	122	126	450	0	----
UNIT 2 - NON-LIQUEFIED ZONE	122	126	0.25P	0	---
UNIT 3A - LIQUIFIED ZONE	126	128	800	0	----
UNIT 3A - NON-LIQUIFIED ZONE	126	128	0	35	35
UNIT 3B - CLAY	128	128	0.25P	0	——
UNIT 3C - LIQUIFIED ZONE	126	128	800	0	——
UNIT 3C - NON-LIQUIFIED ZONE	126	128	0	35	50

P IS THE EFFECTIVE OVERBURDEN PRESSURE

ITCHYARD SECTION

ES OF LIQUEFACTION



'S ALONG SECTION (FT)

U.S. ARMY ENGINEER DISTRICT
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NASHVILLE, TENNESSEE

Drawn By:

Checked By:

Approved By:

Date:

Scale:

Sheet of

Record Drawing as constructed dated

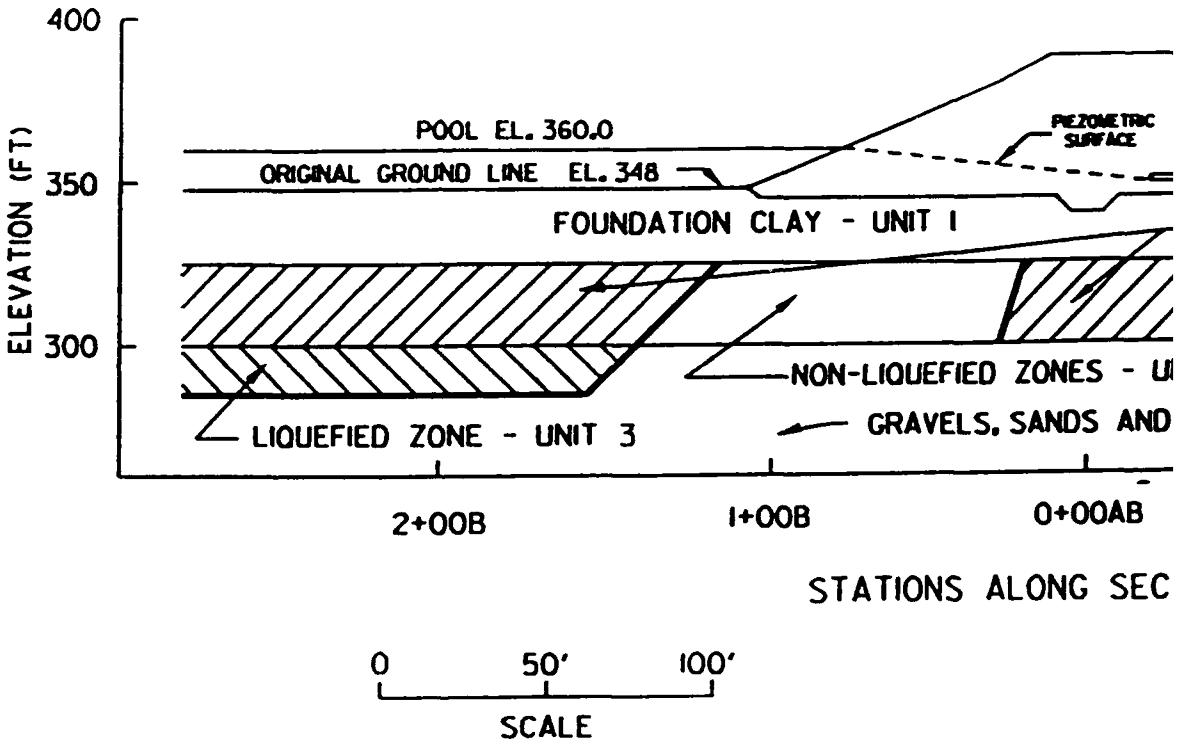
Drawing Number

Computer Aided Design & Drafting

2 | 1

rthquake strengths for switchyard section.

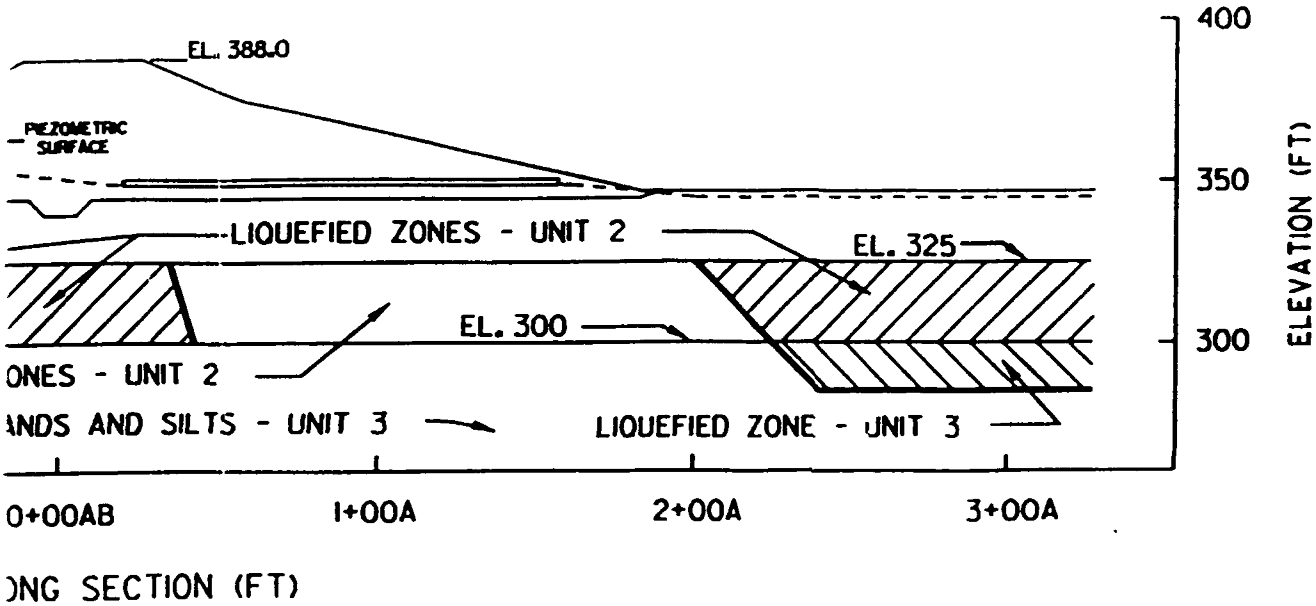
ZONES OF LIQU



- LIQUEFACTION

STABILITY ANALYSIS PARAMETERS

MATERIAL	UNIT WEIGHT γ_m	UNIT WEIGHT γ_{sat}	C (PSF)	φ	% Ru
COMPACTED FILL	126	128	800	18	---
UNIT 1 CLAY	115	125	960	12	---
LIQUEFIED ZONE - UNIT 2	122	126	700	0	---
UNIT 3 SANDS	126	128	0	35	50
LIQUEFIED ZONE - UNIT 3	126	128	800	0	---



U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NASHVILLE, TENNESSEE

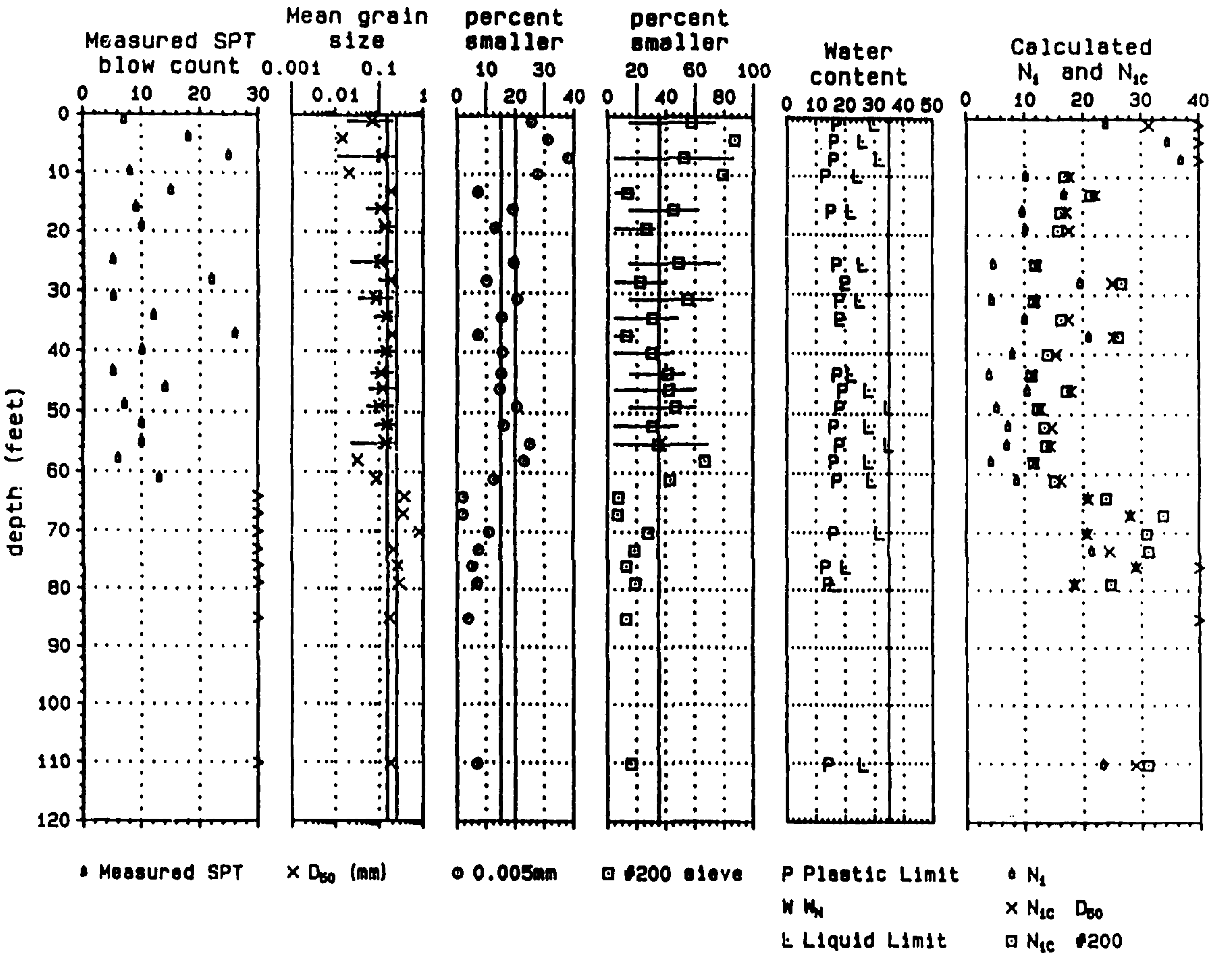
BARKLEY DAM SEISMIC ANALYSIS
MAIN EMBANKMENT
ZONES OF LIQUEFACTION

Drawn By:
Checked By:
Approved By:
Date:
Scale:
Sheet of
Record Drawing as constructed dated
Drawing Number

e strengths for Main Embankment Section.

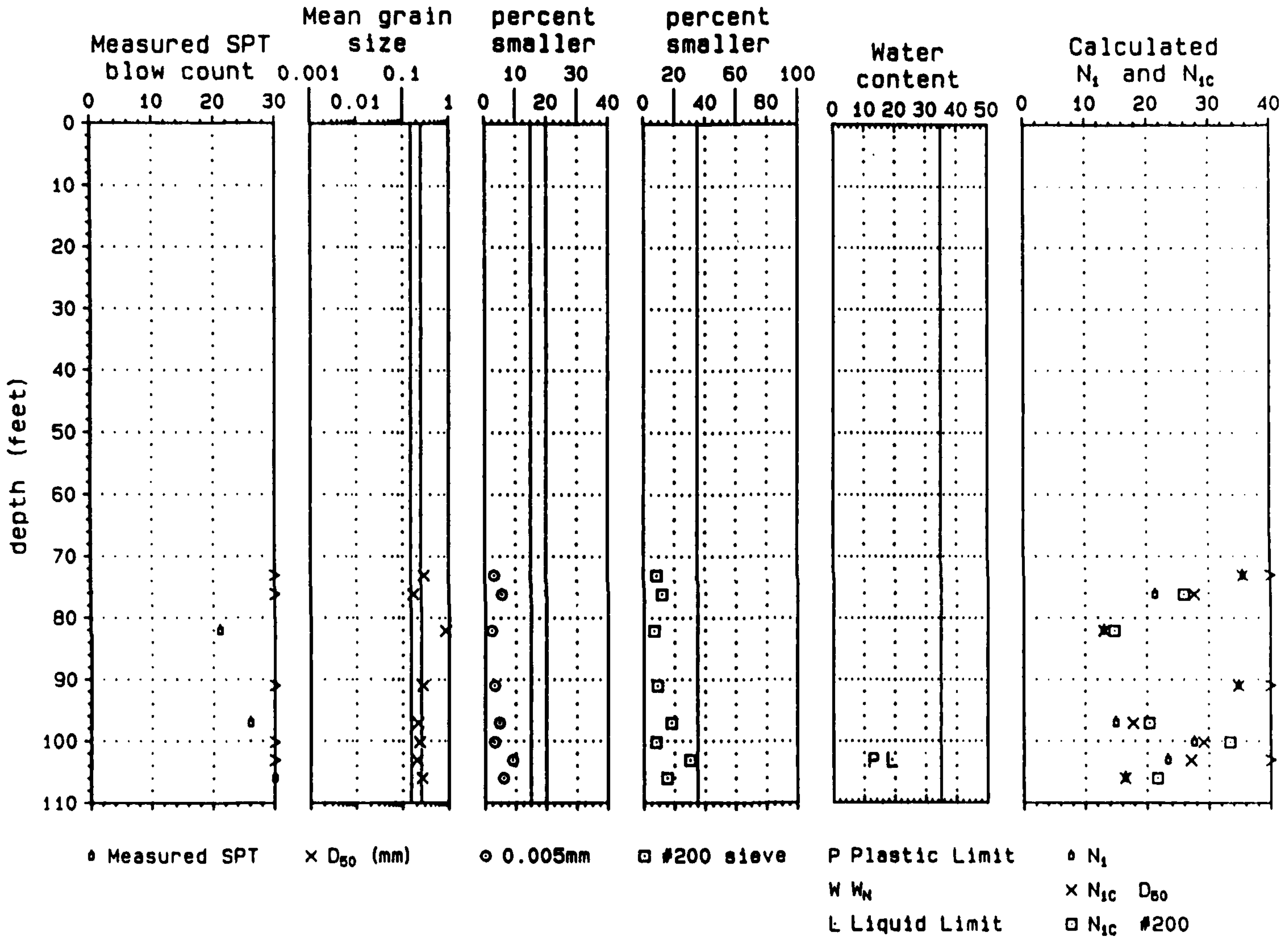
APPENDIX A:

SPT DATA PLOTS



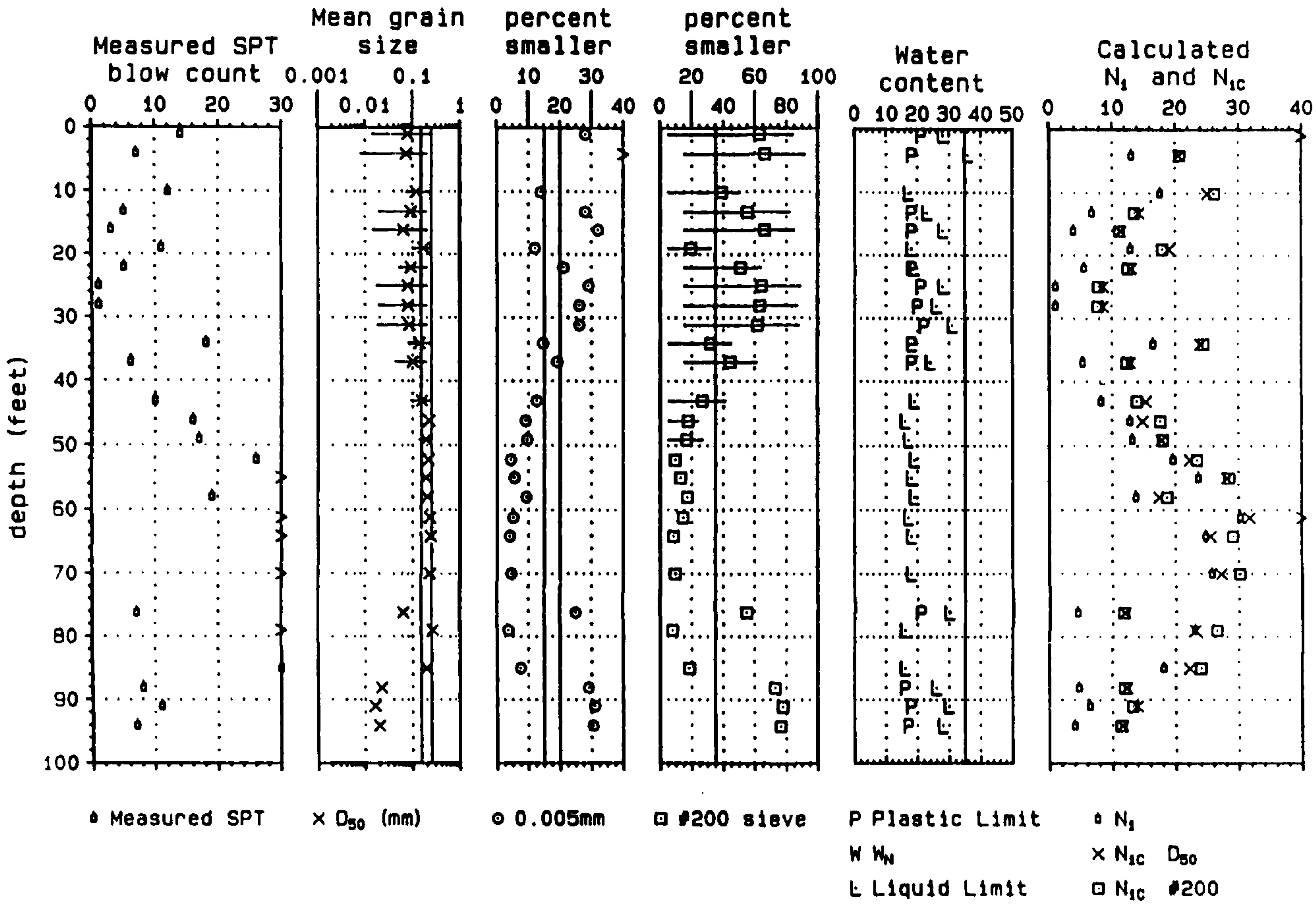
BEQ-01
Barkley dam

SPT Liquefaction Analysis



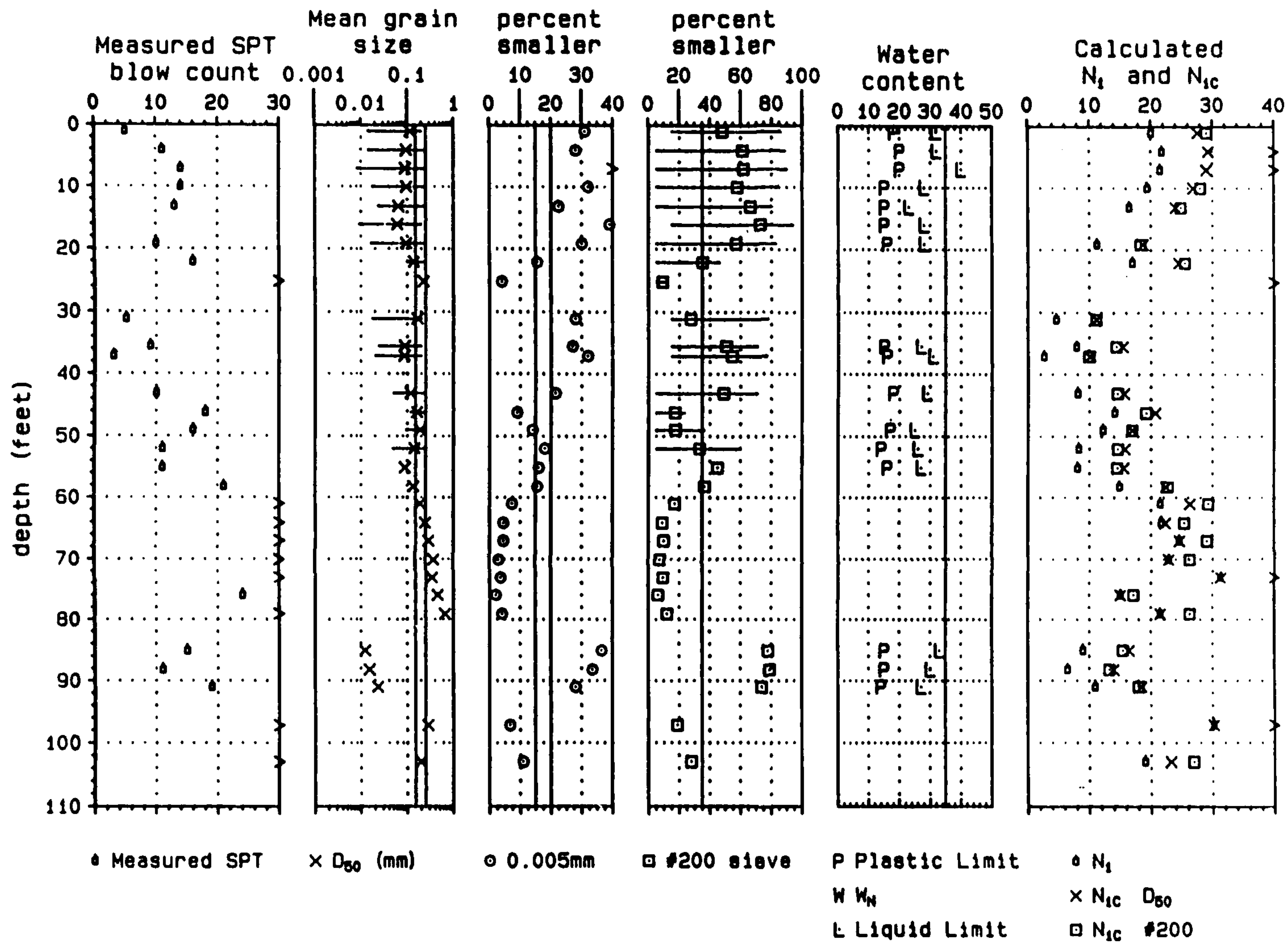
BEQ-02
Barkley dam

SPT Liquefaction Analysis



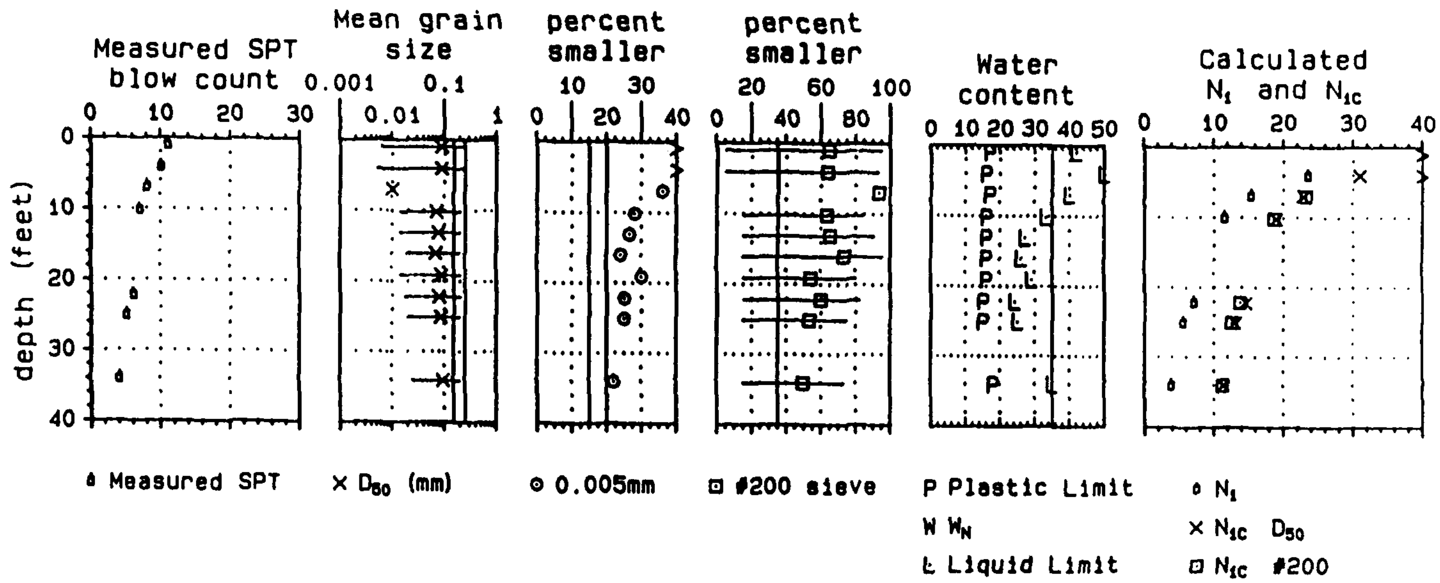
BEQ-03
Barkley dam

SPT Liquefaction Analysis



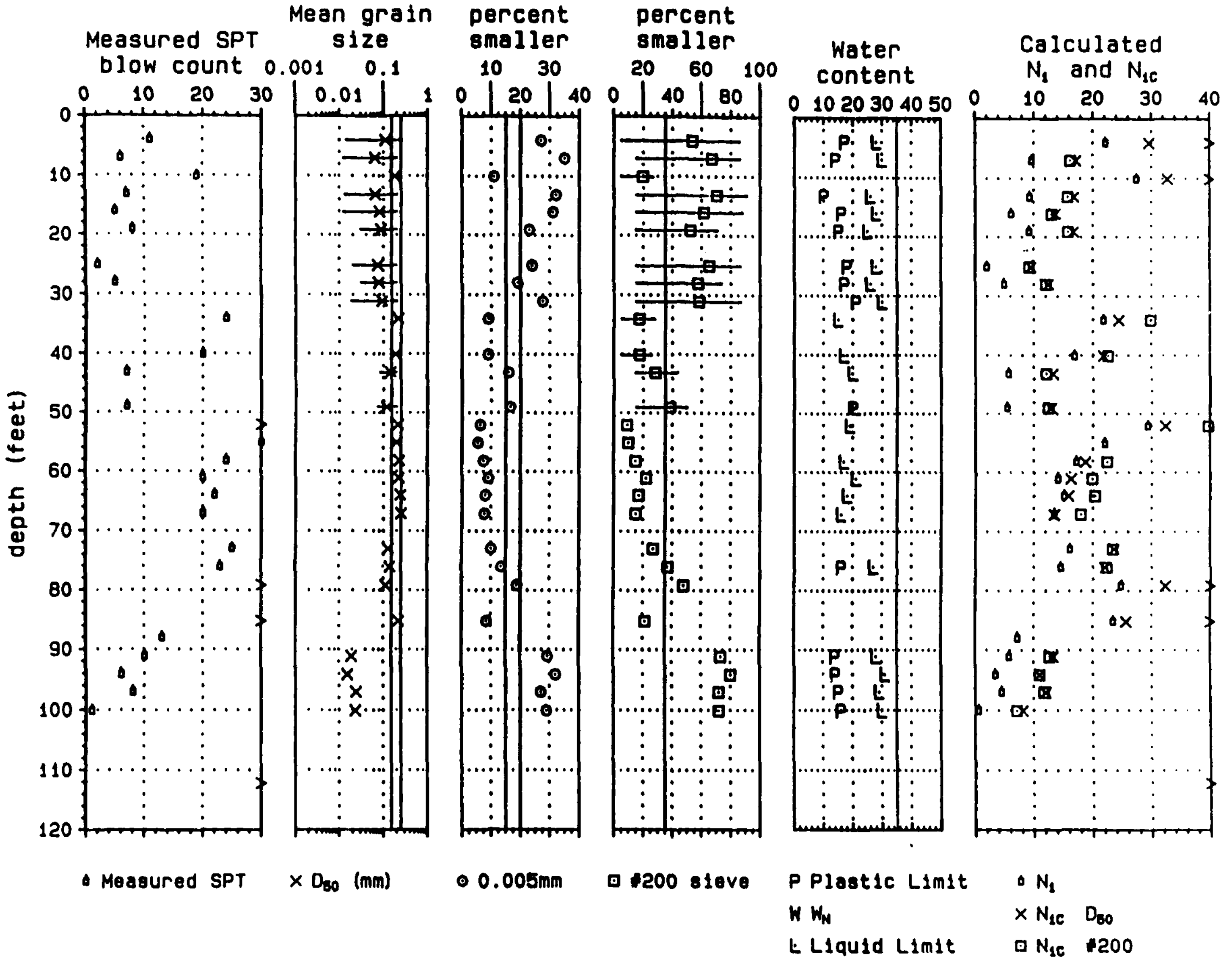
BEQ-04
Barkley dam

SPT Liquefaction Analysis



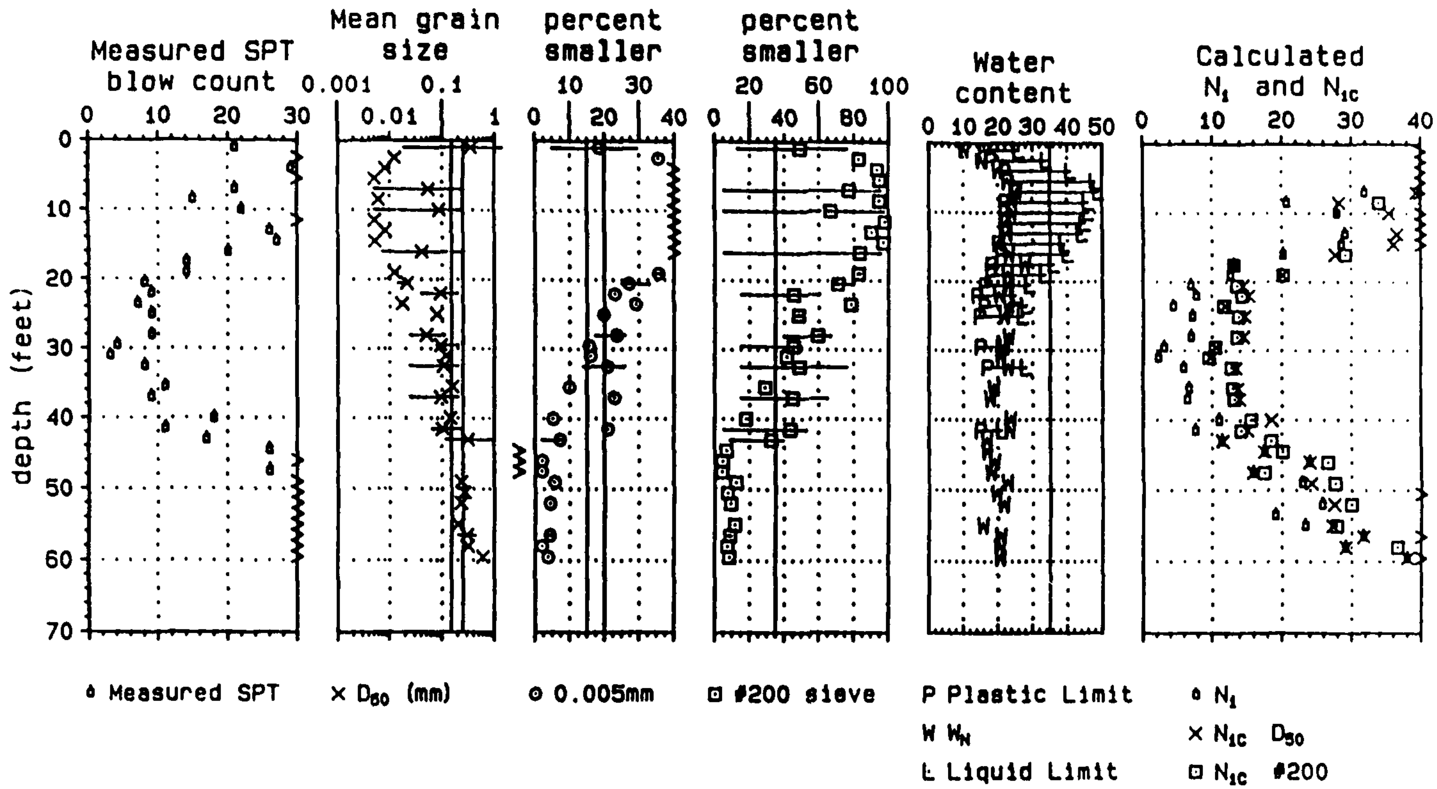
BEQ-05
Barkley dam

SPT Liquefaction Analysis



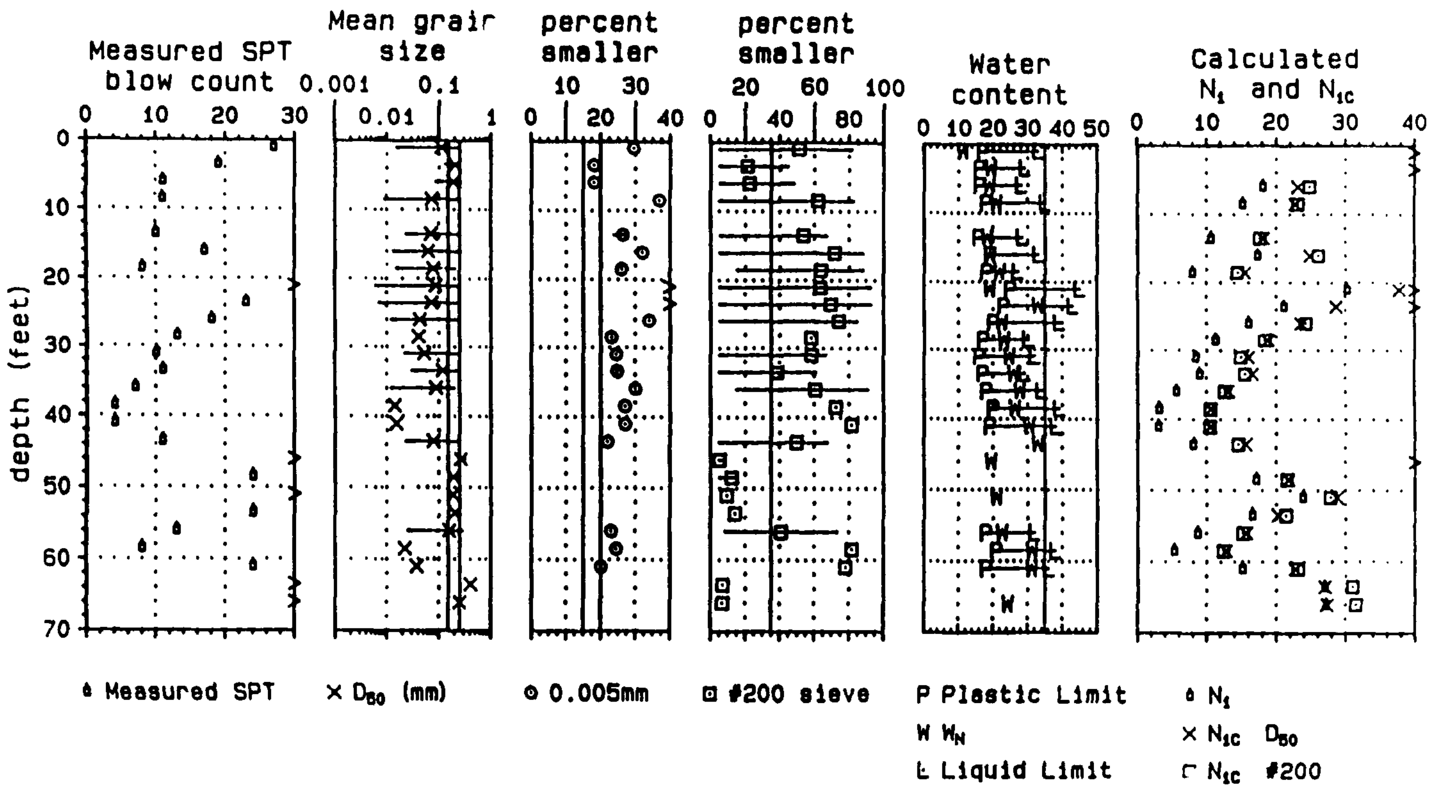
BEQ-06
Barkley dam

SPT Liquefaction Analysis



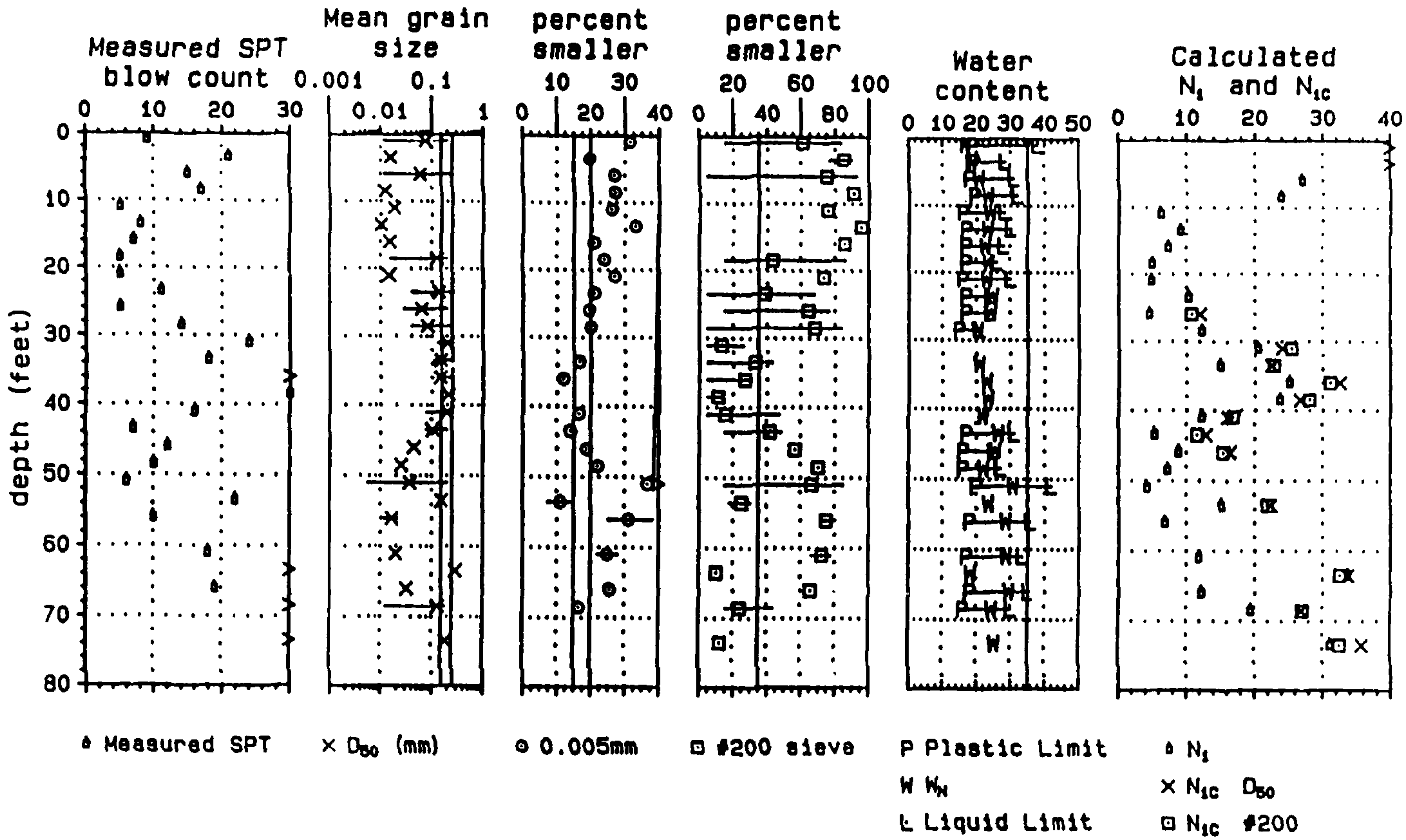
BEQ-07
Barkley dam

SPT Liquefaction Analysis



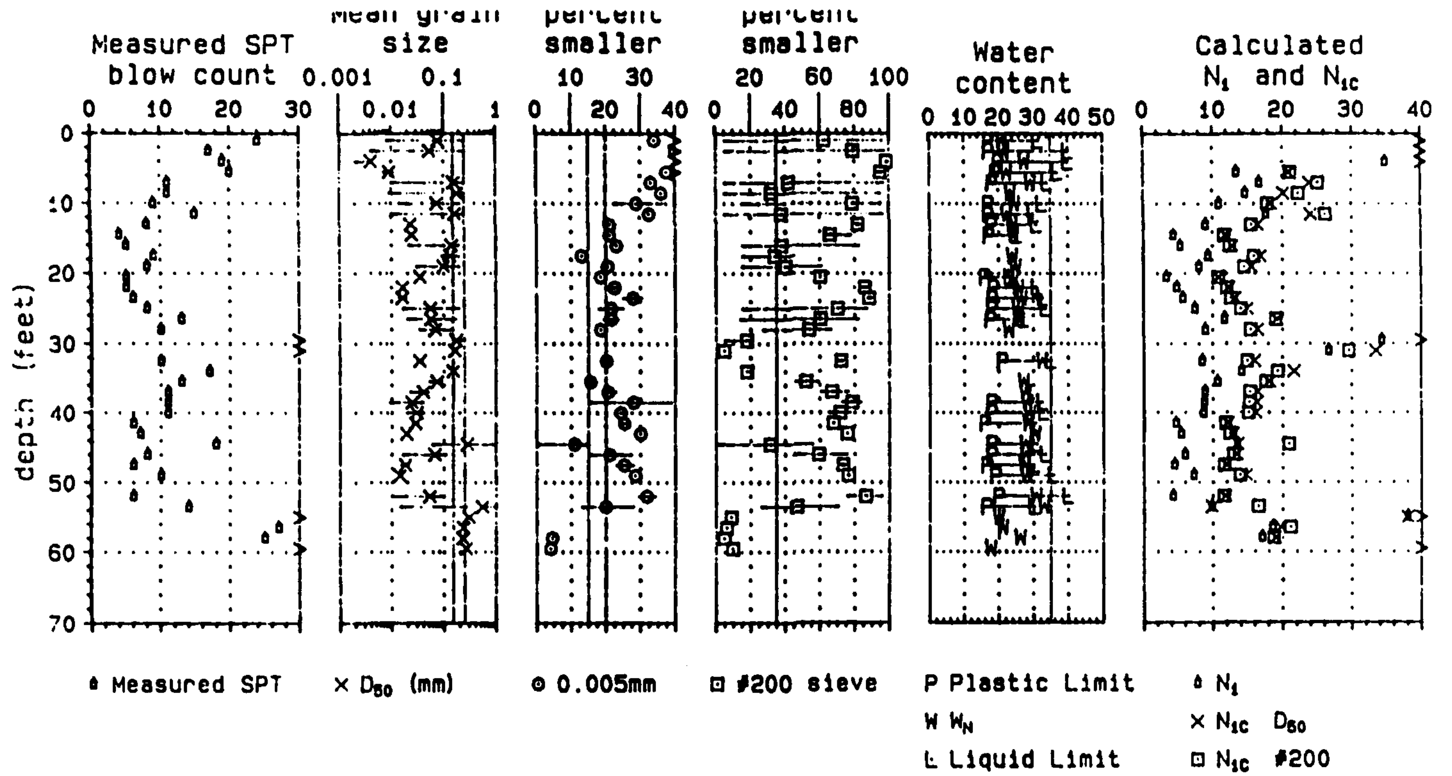
BEQ-08
Barkley dam

SPT Liquefaction Analysis



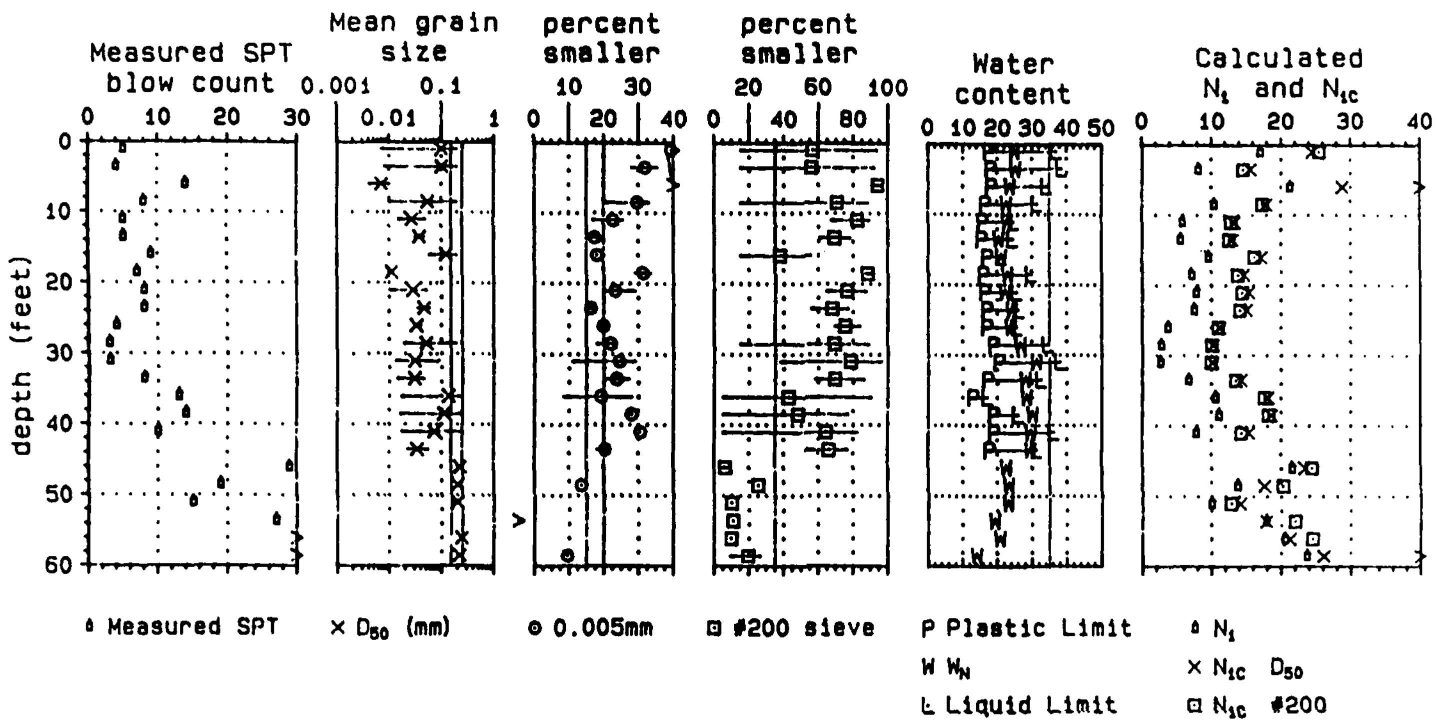
SPT Liquefaction Analysis

BEQ-09
Barkley dam



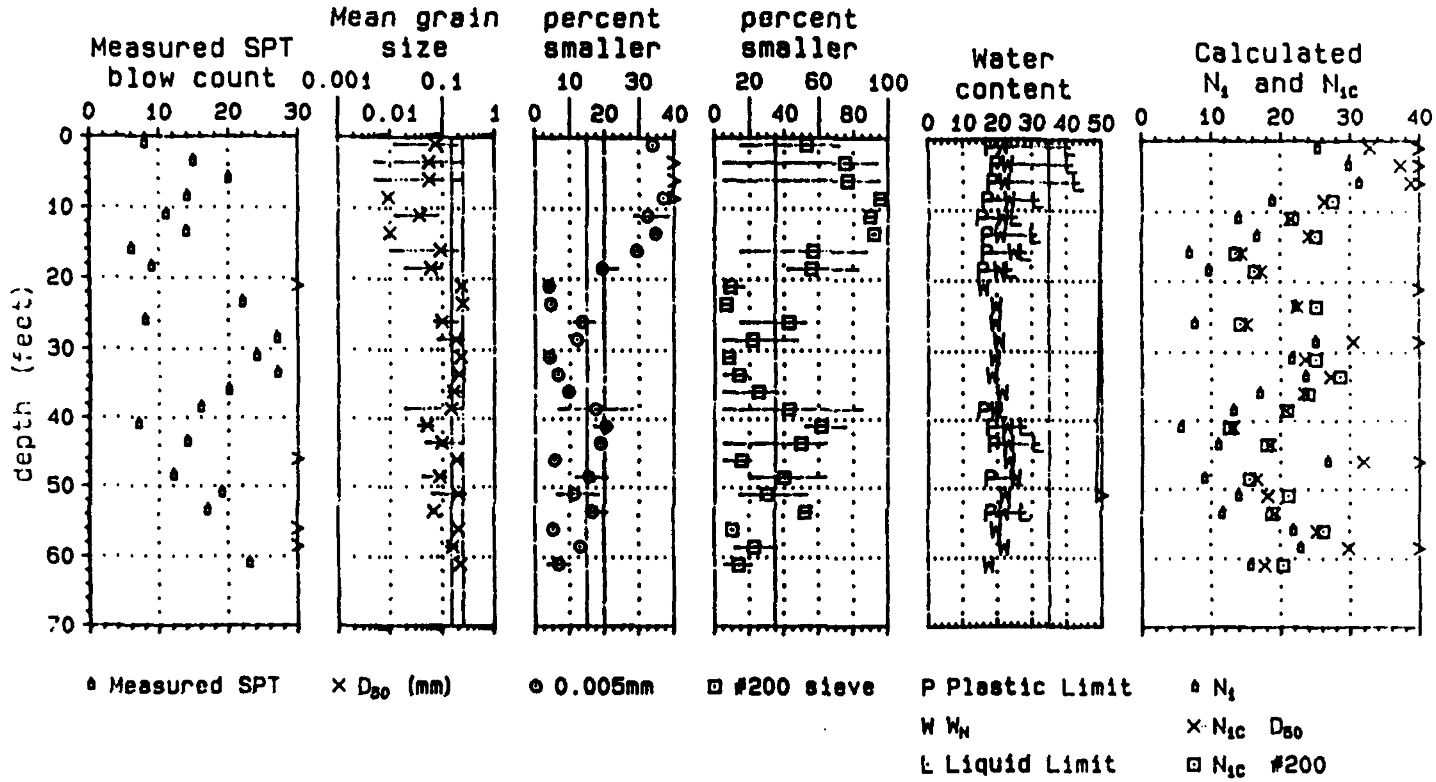
BEQ-10
Barkley dam

SPT Liquefaction Analysis



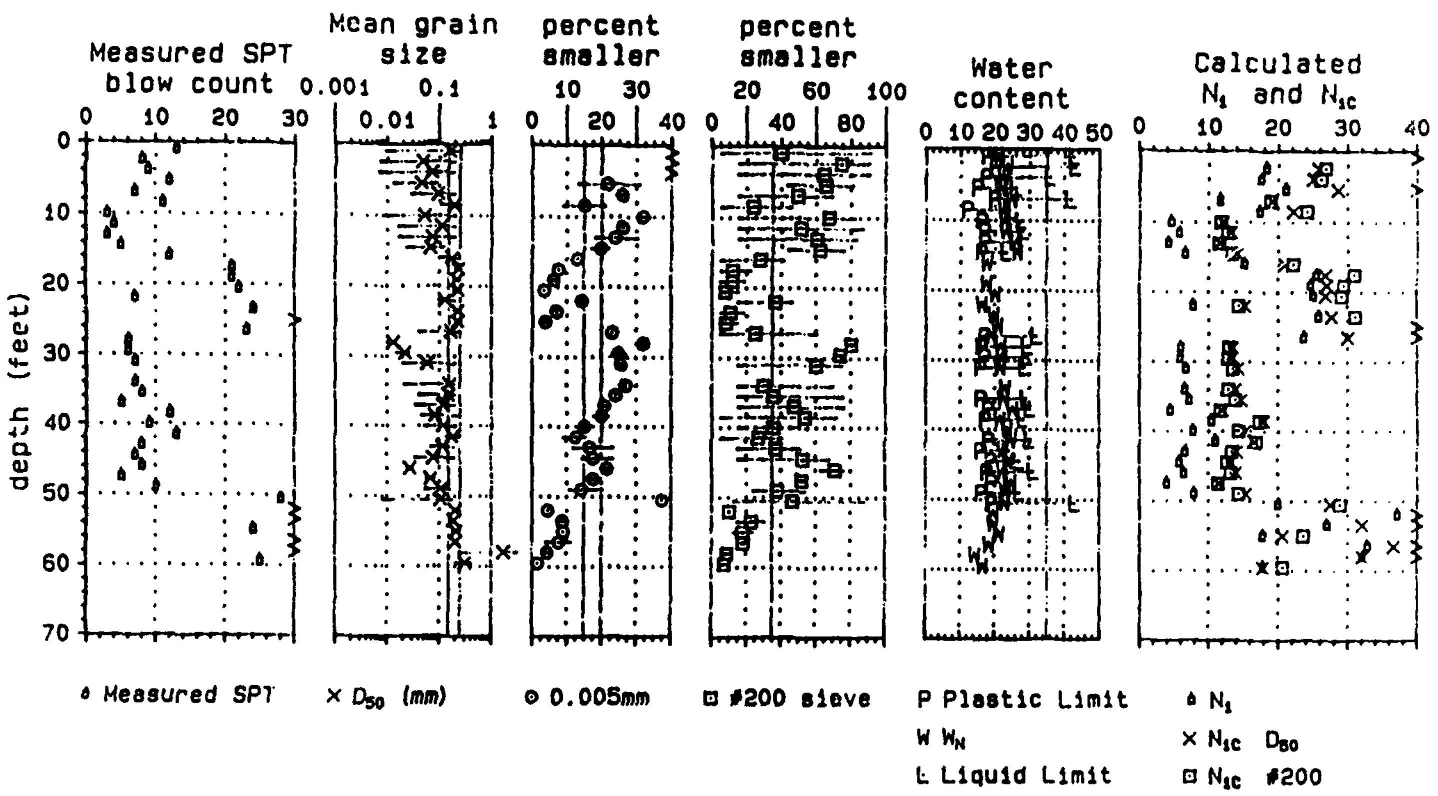
BEQ-11
Barkley dam

SPT Liquefaction Analysis



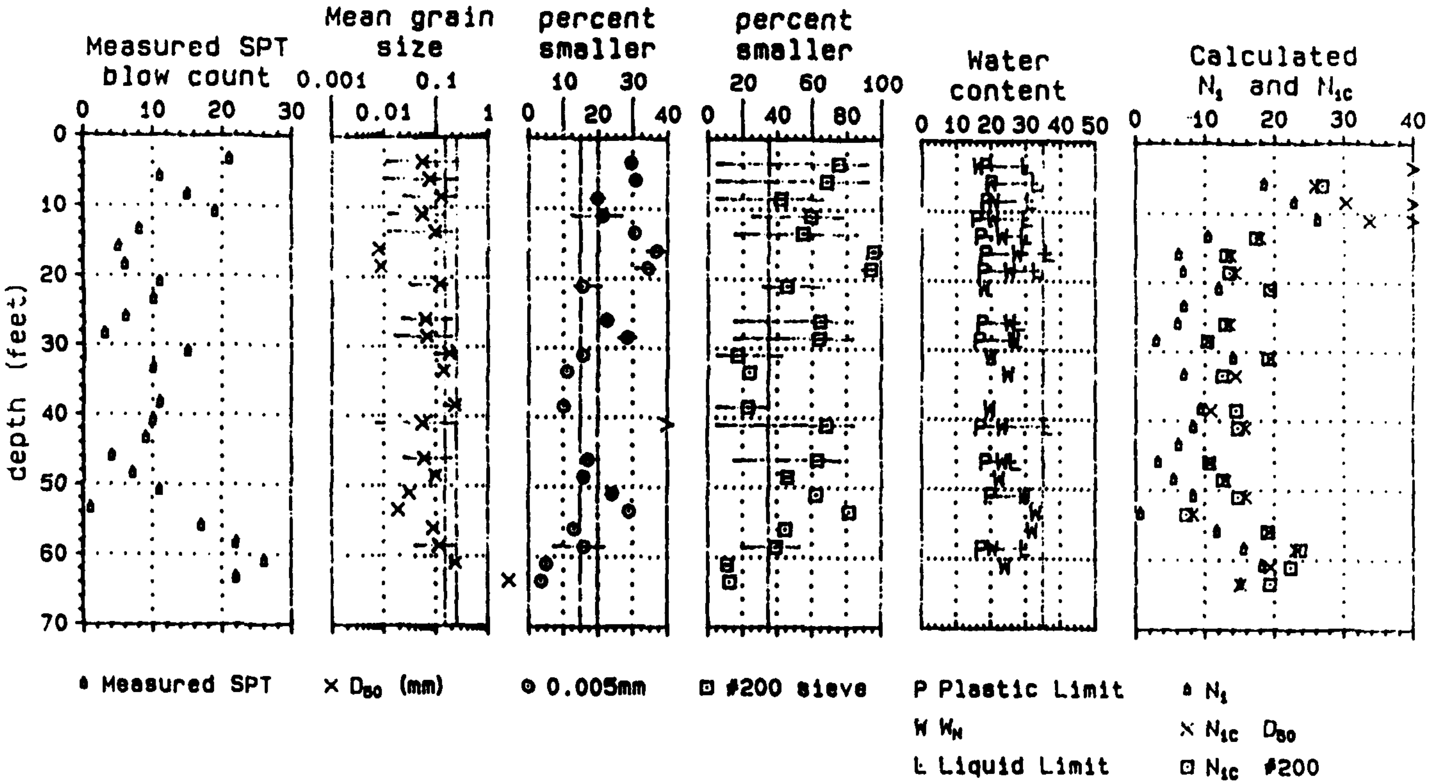
BEQ-12
Barkley dam

SPT Liquefaction Analysis



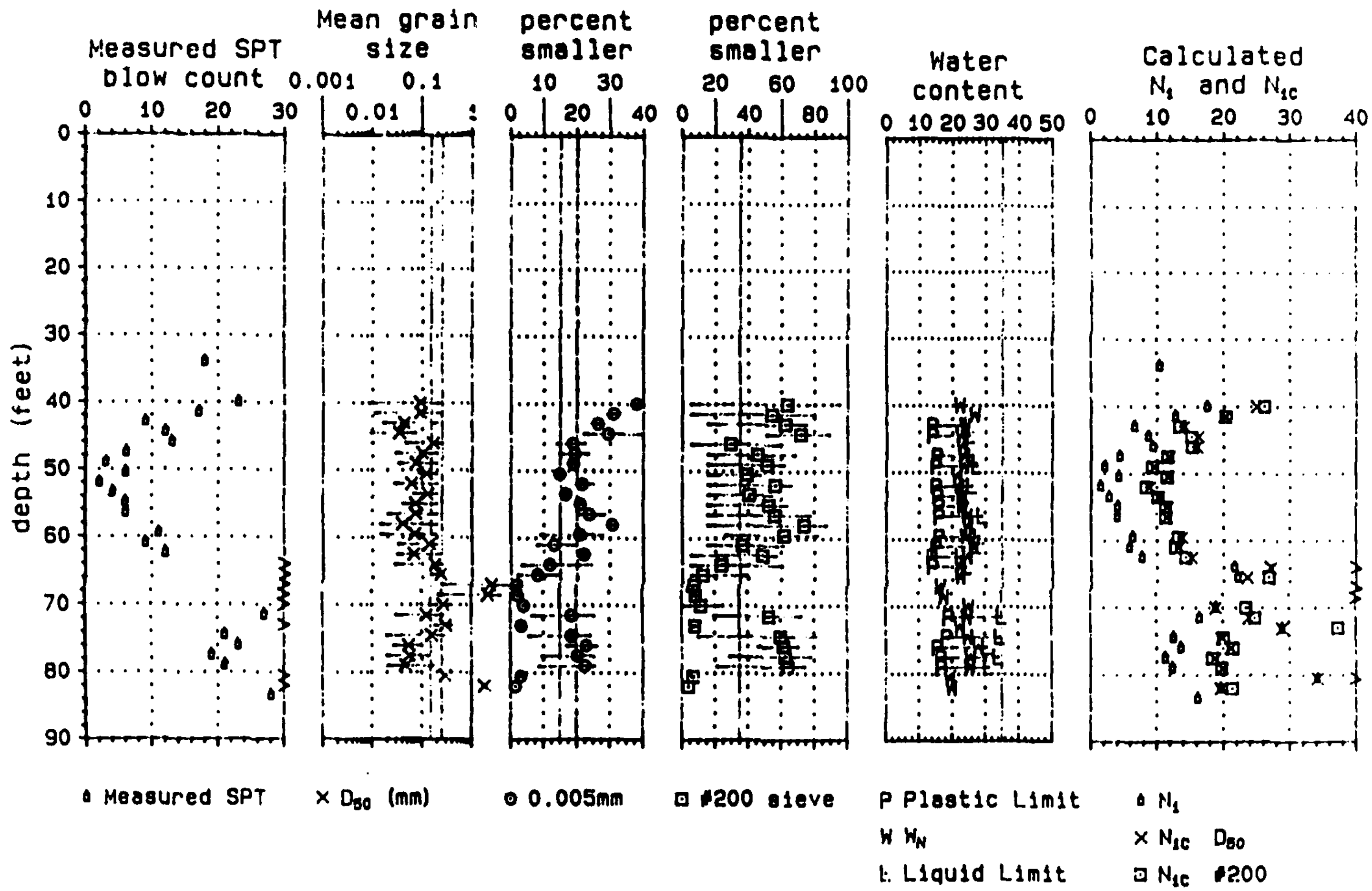
BEQ-13
Barkley dam

SPT Liquefaction Analysis



BEQ-14
Barkley dam

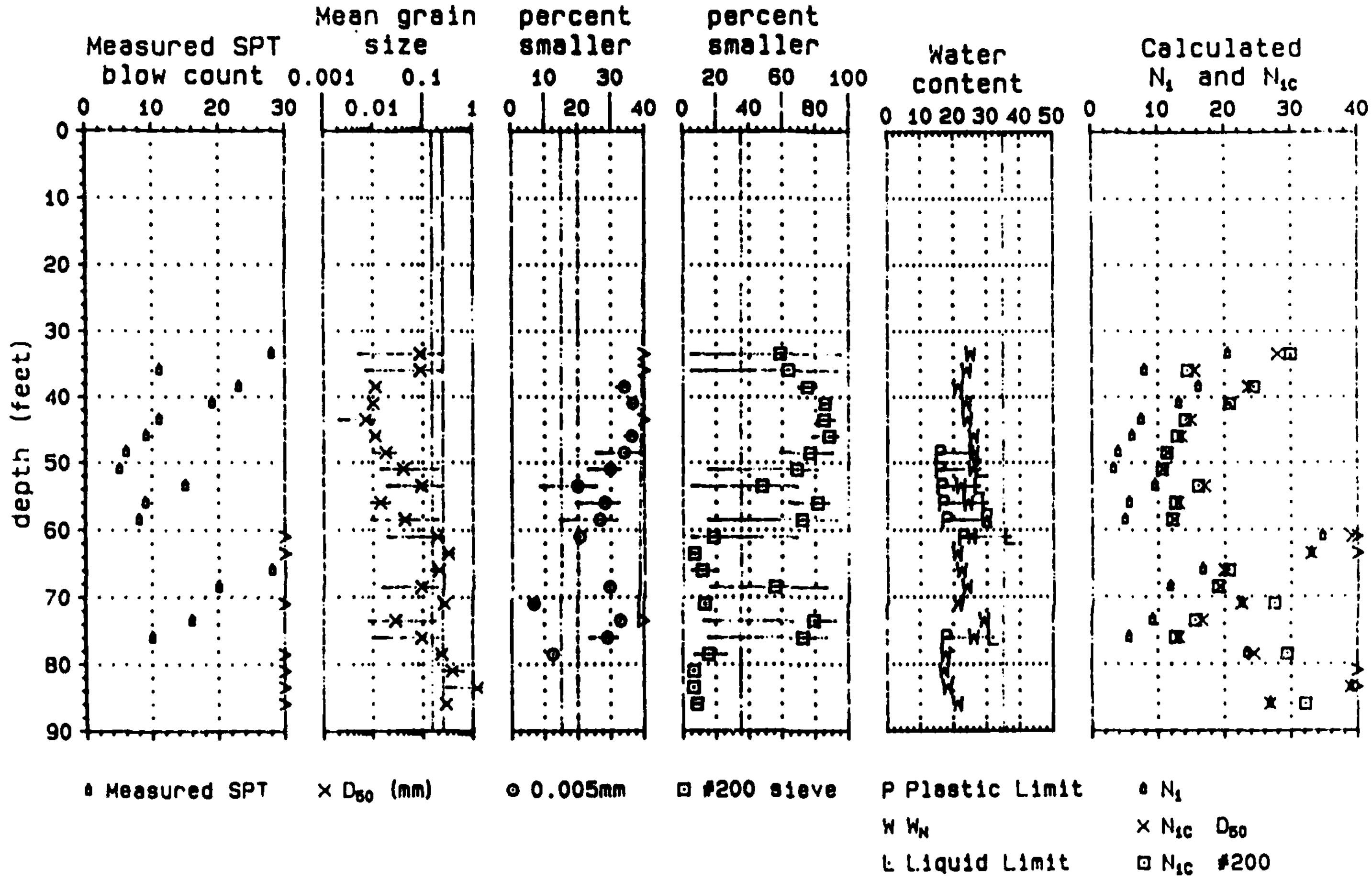
SPT Liquefaction Analysis



BEQ-16
Barkley dam

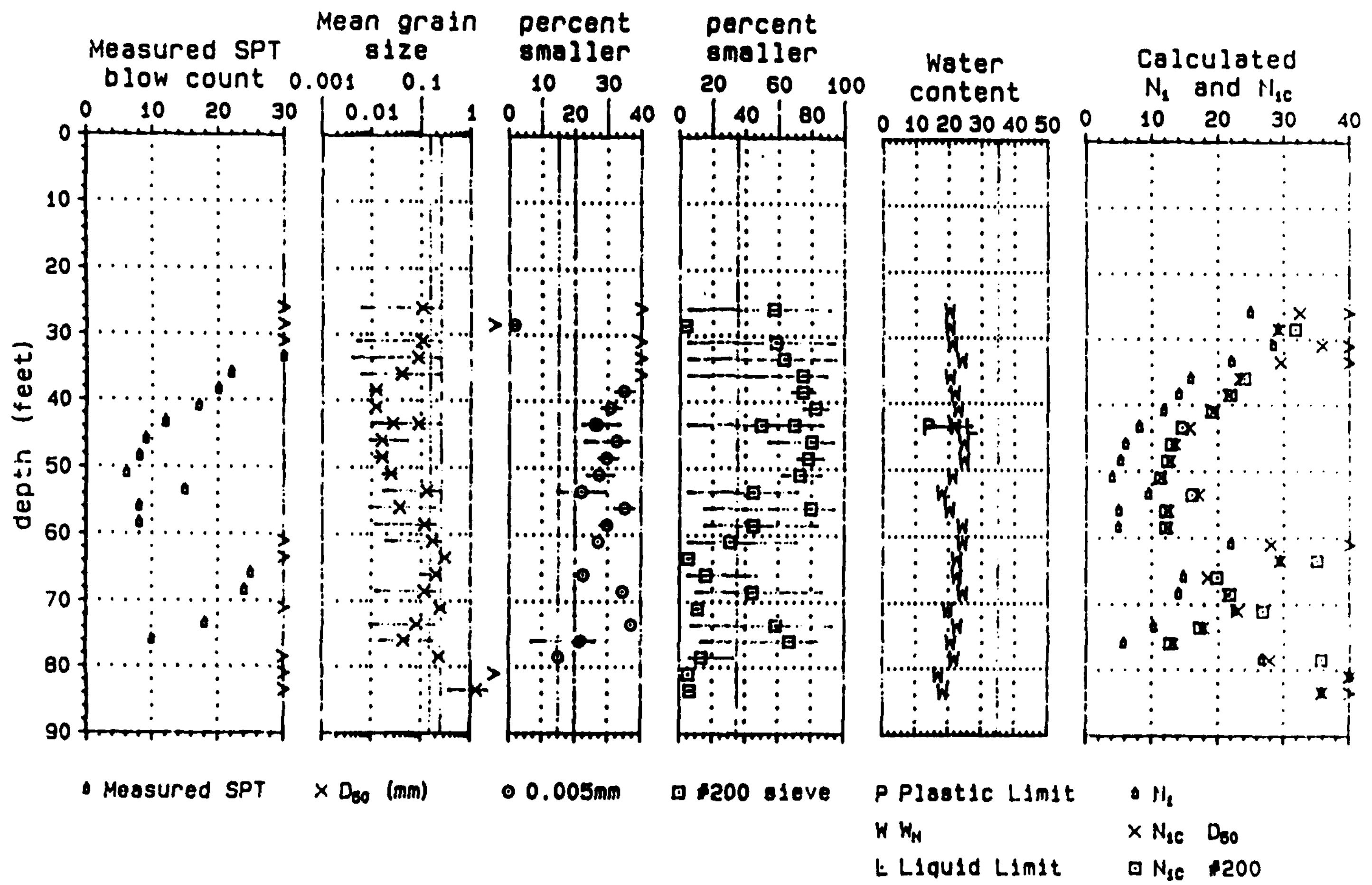
SPT Liquefaction Analysis

ASO xyplot



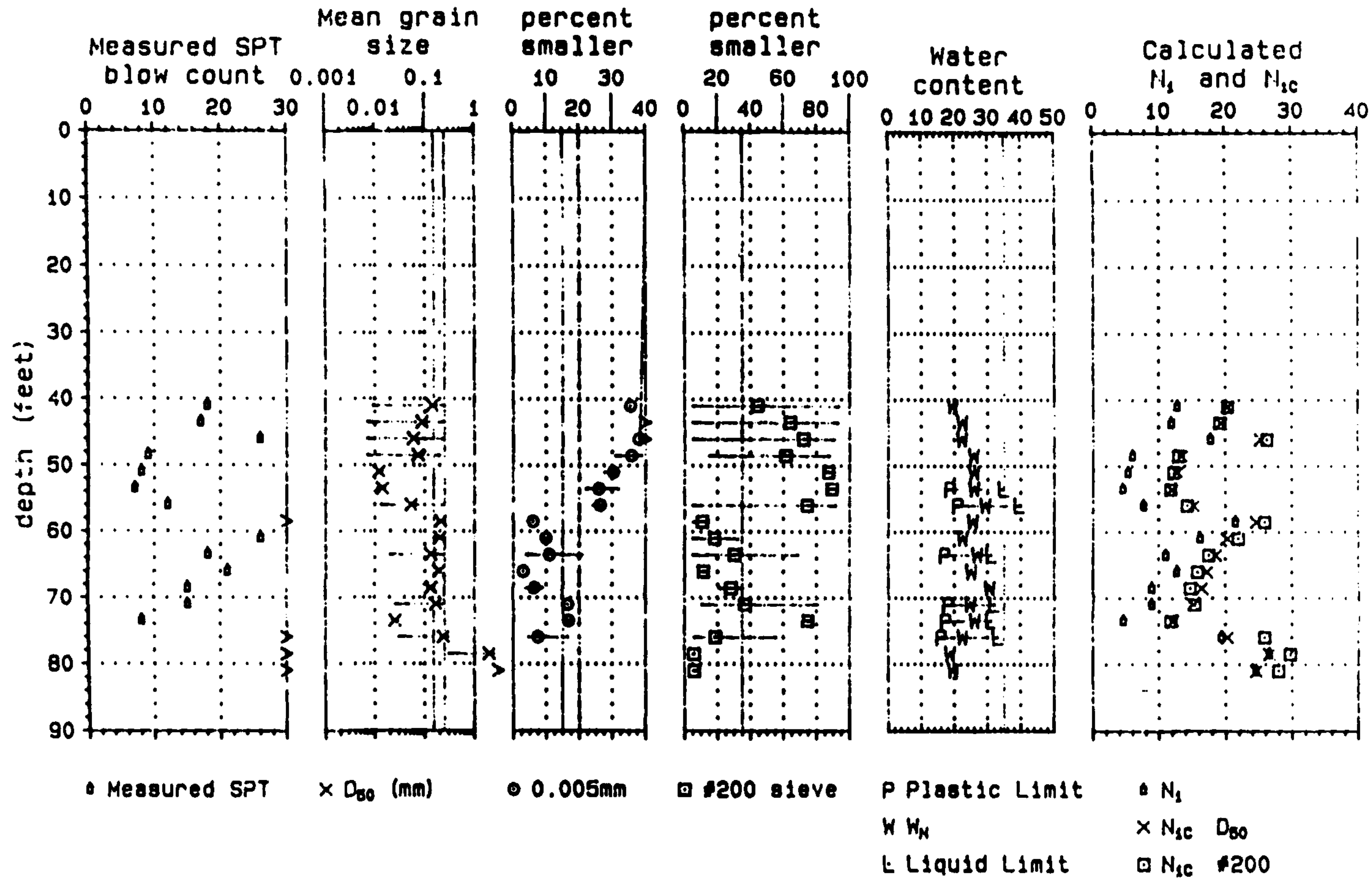
BEQ-17
Barkley dam

SPT Liquefaction Analysis



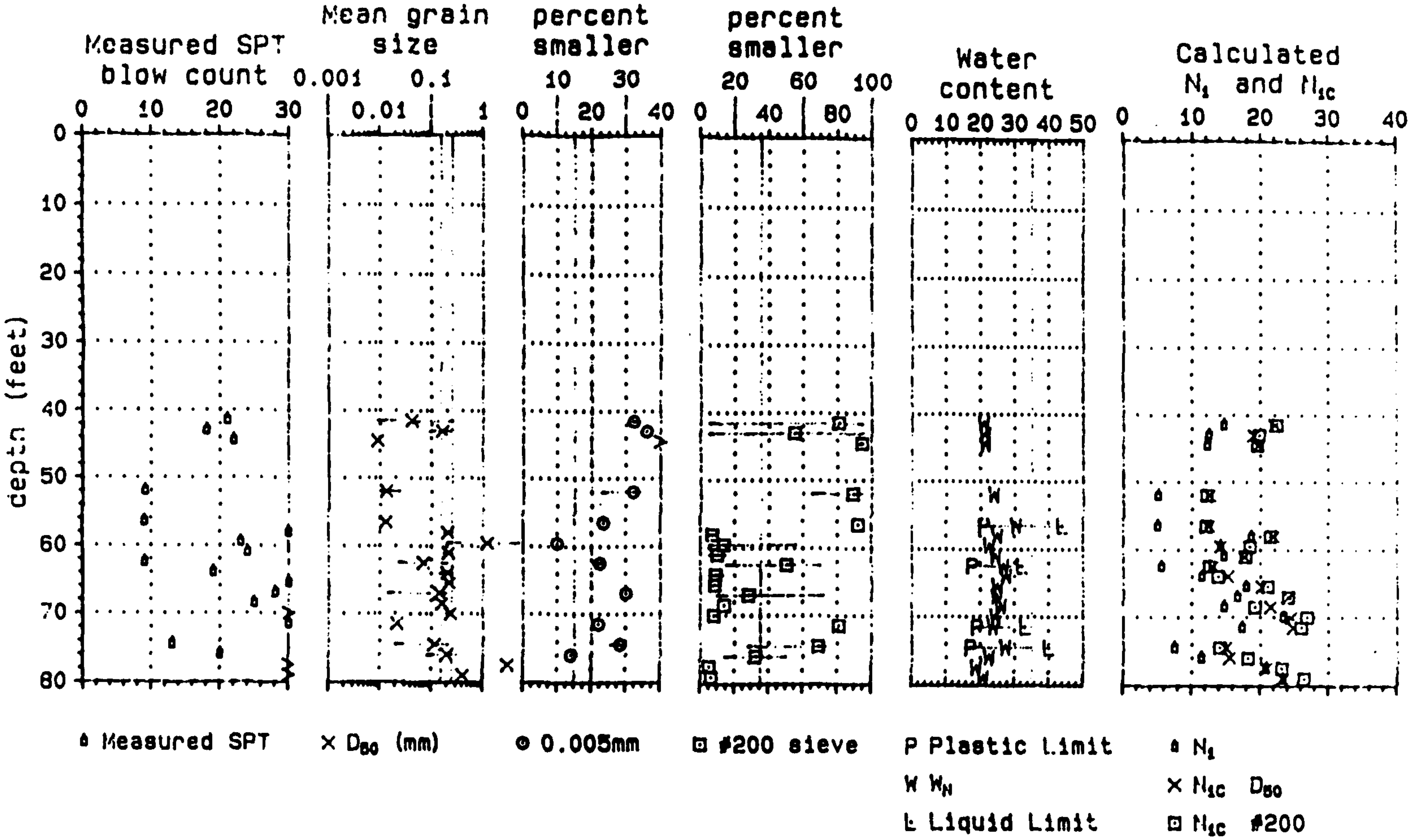
BEQ-18
Barkley dam

SPT Liquefaction Analysis



BEQ-19
Barkley dam

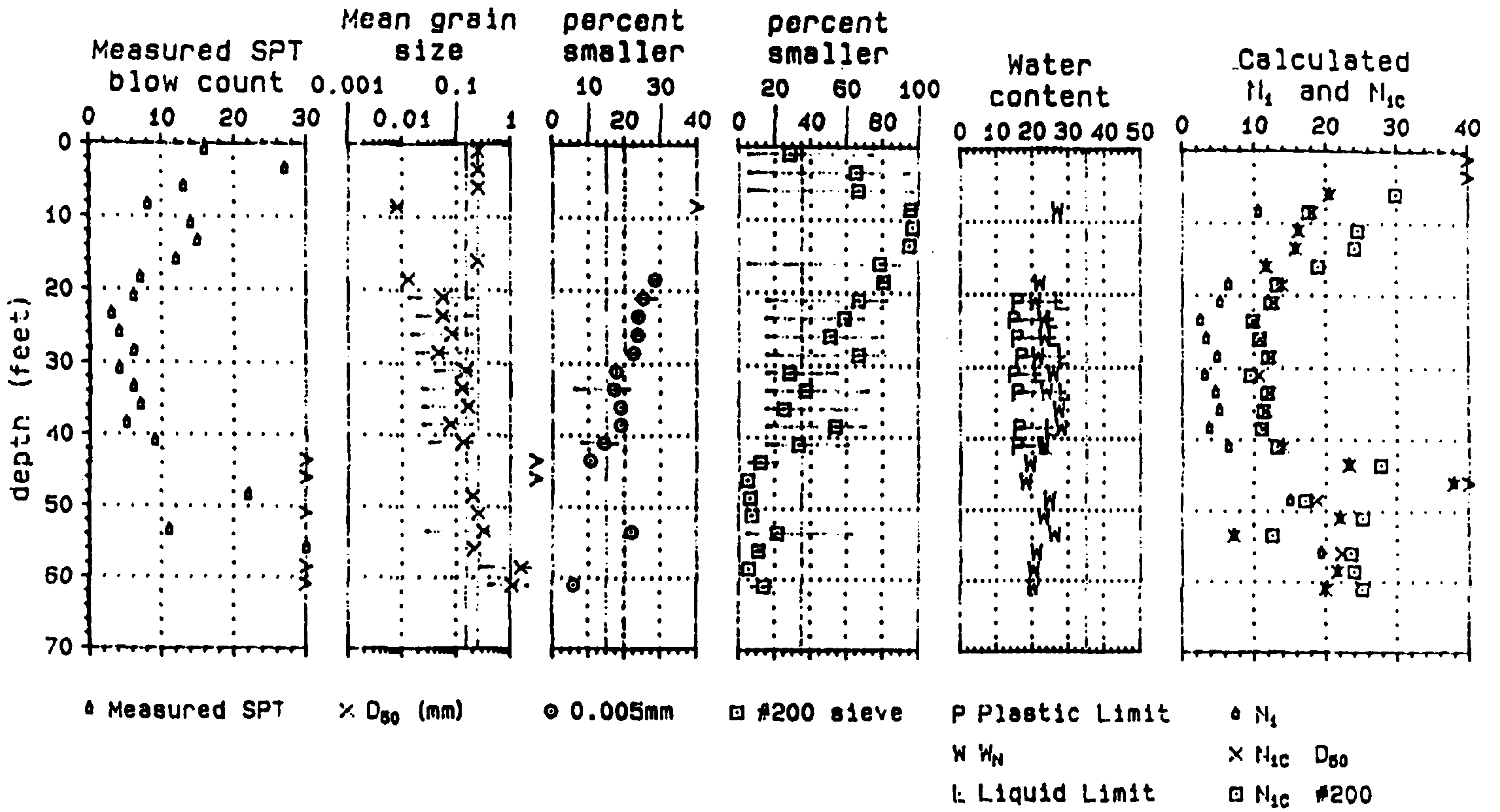
SPT Liquefaction Analysis



BEQ-20
Barkley dam

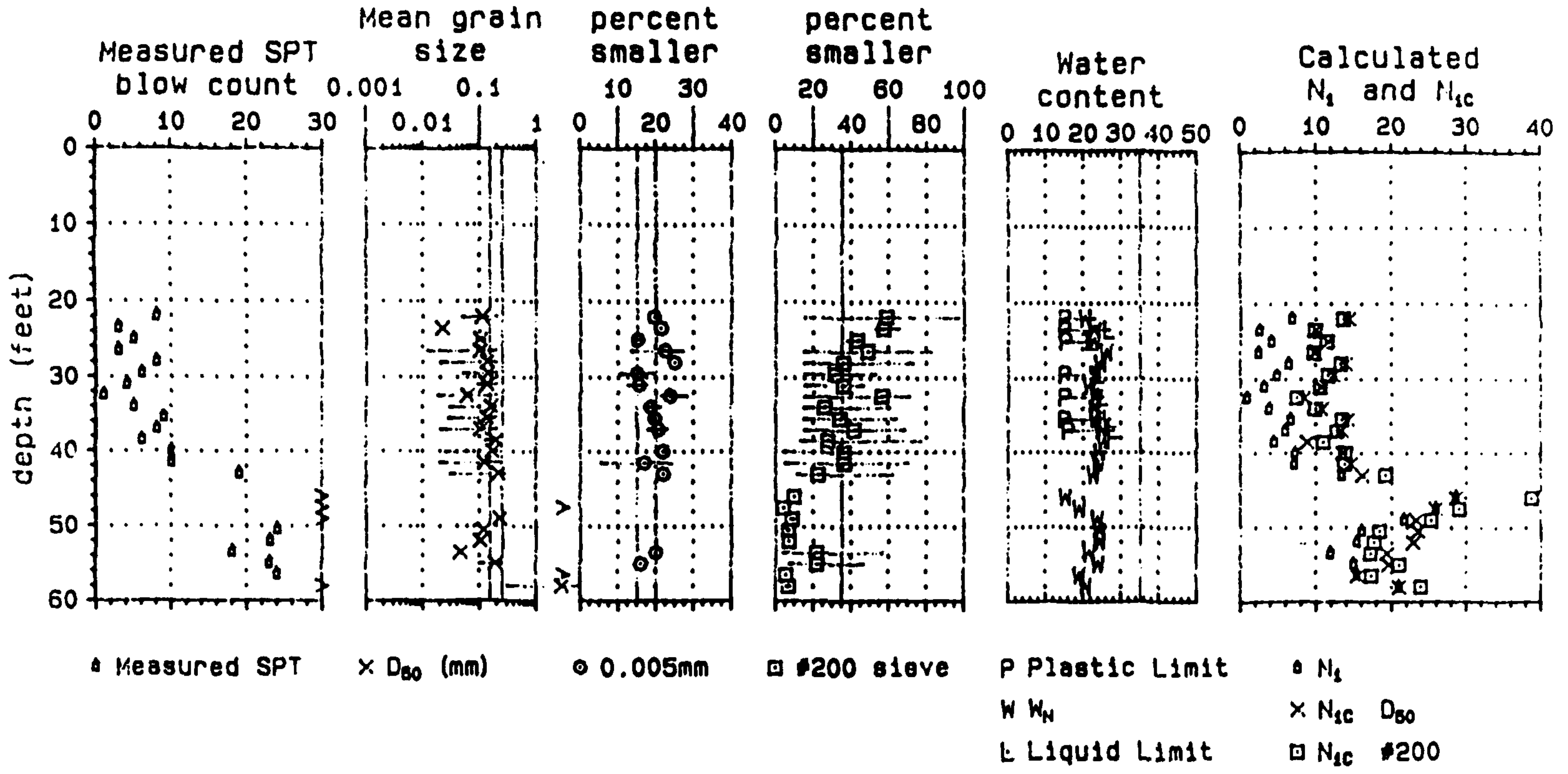
SPT Liquefaction Analysis

ASO xyplot



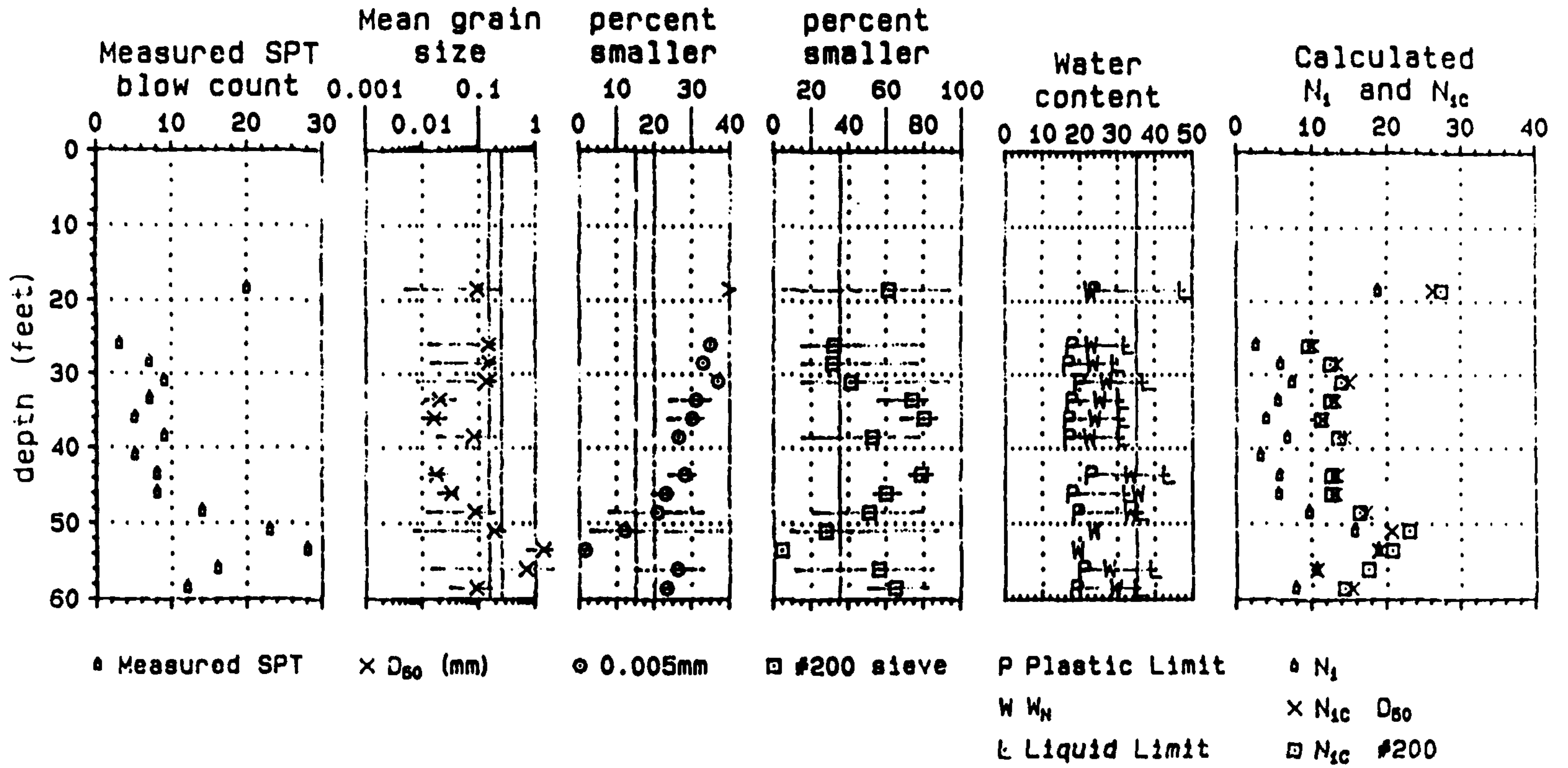
BEQ-21
Barkley dam

SPT Liquefaction Analysis



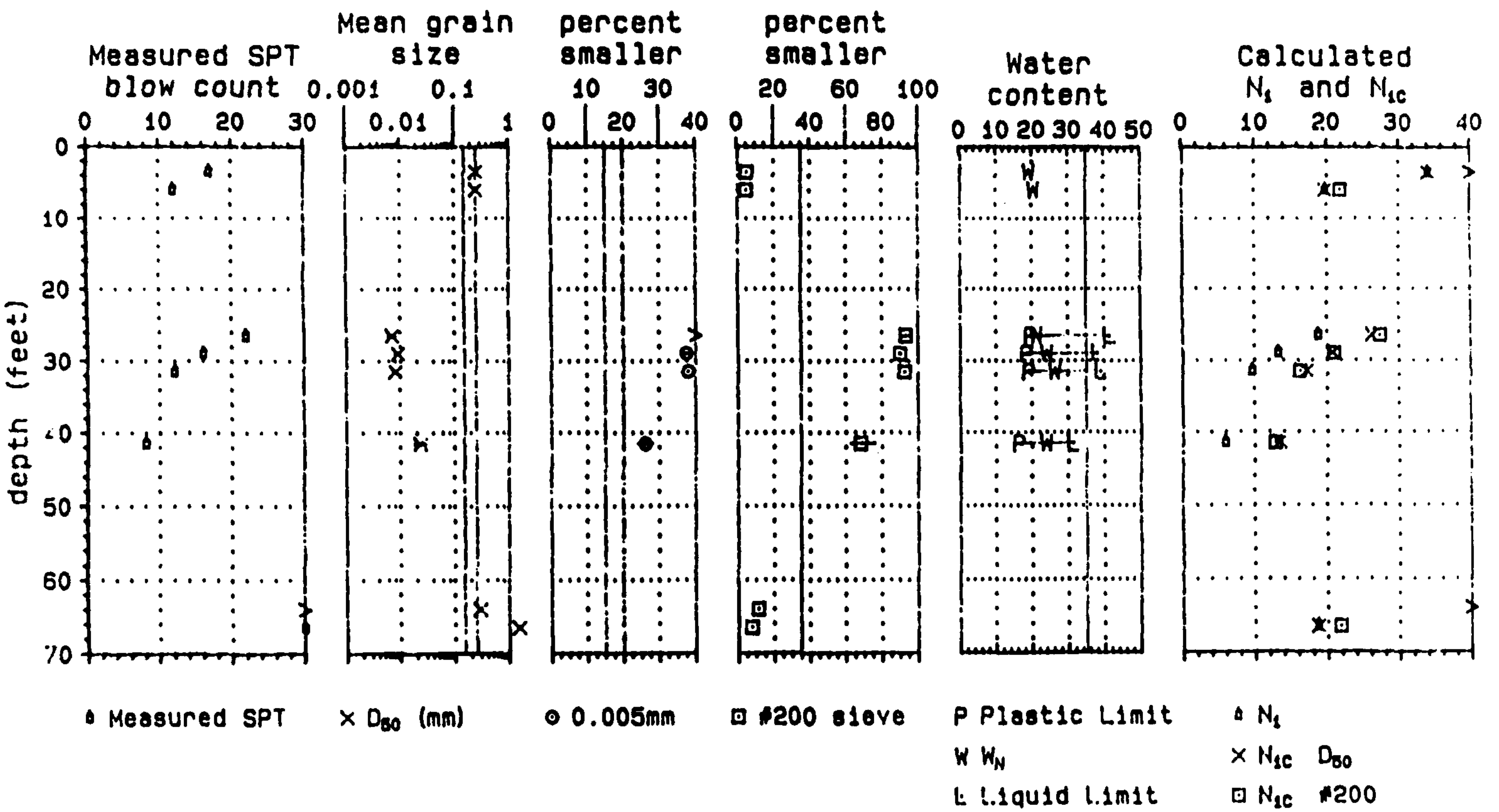
BEQ-22
Barkley dam

SPT Liquefaction Analysis



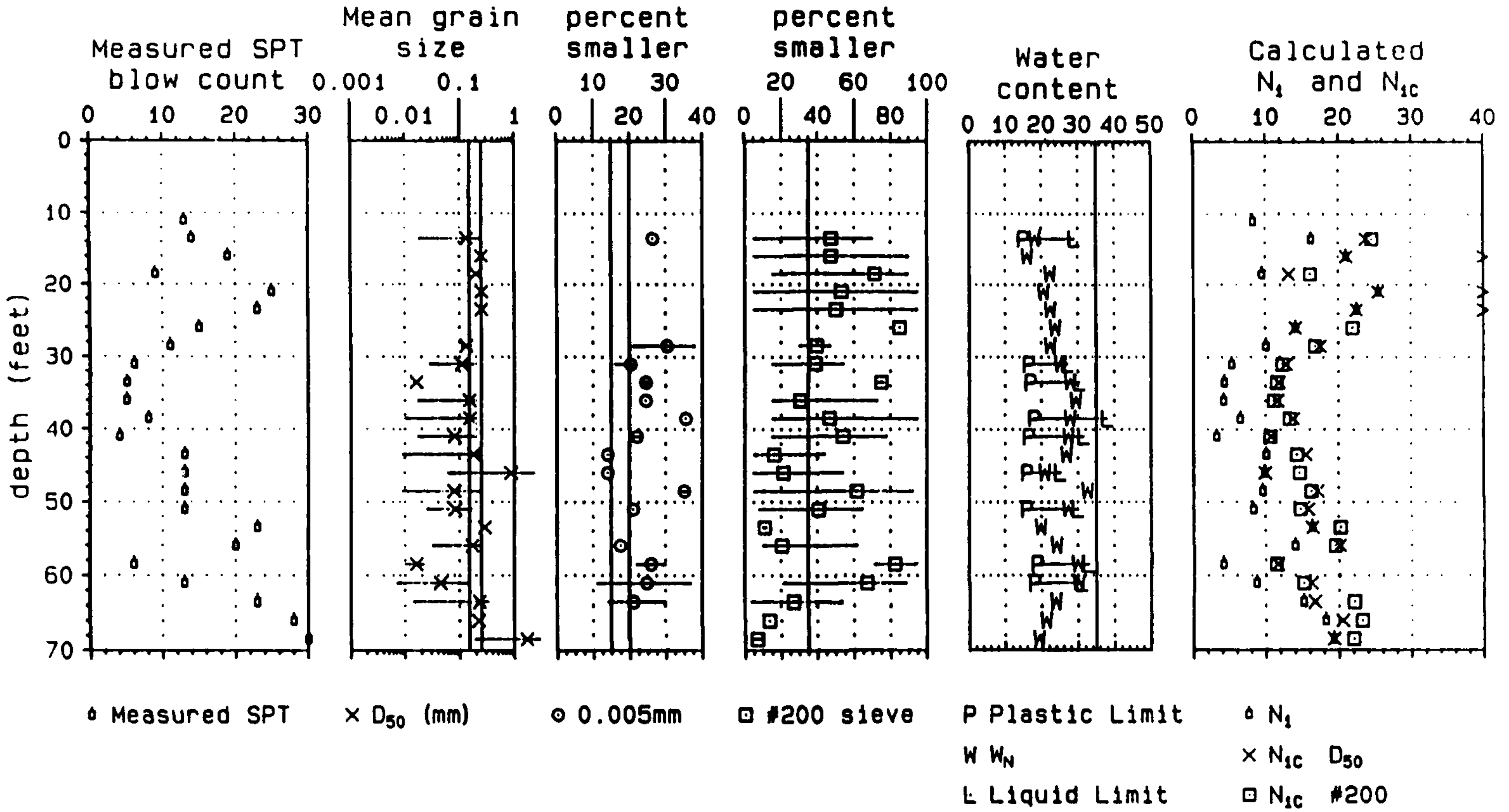
BEQ-23
Barkley dam

SPT Liquefaction Analysis



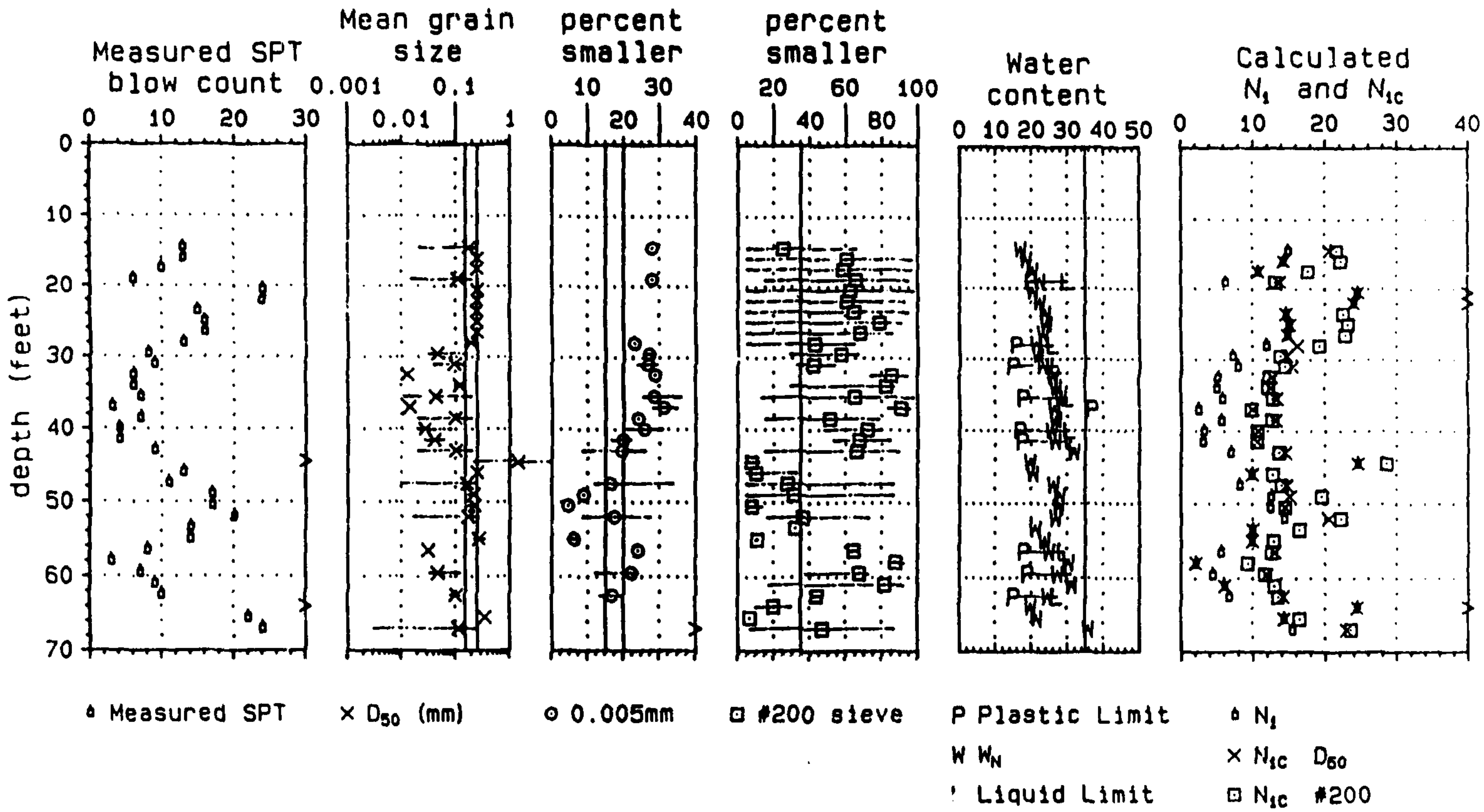
BEQ-24
Barkley dam

SPT Liquefaction Analysis



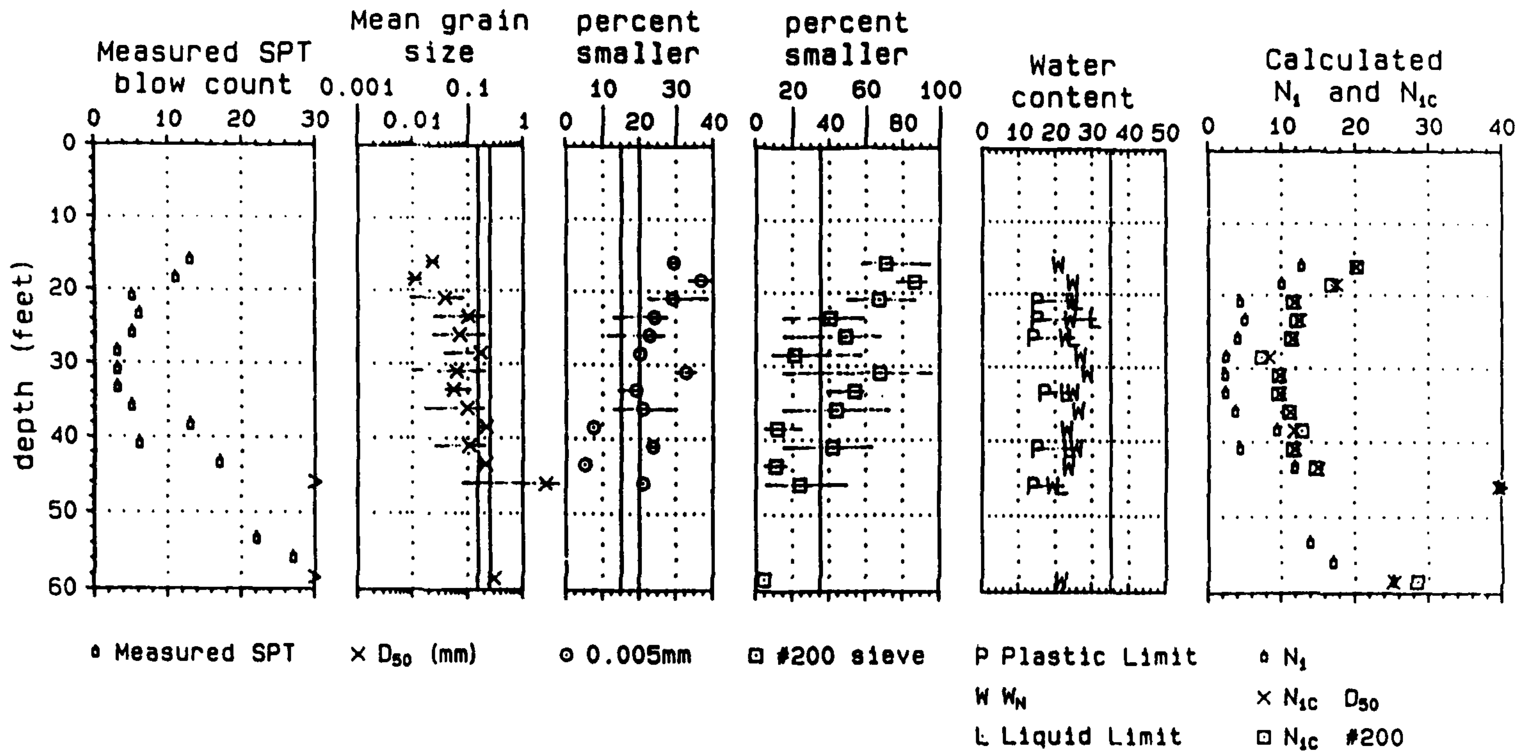
BEQ-25
Barkley dam

SPT Liquefaction Analysis



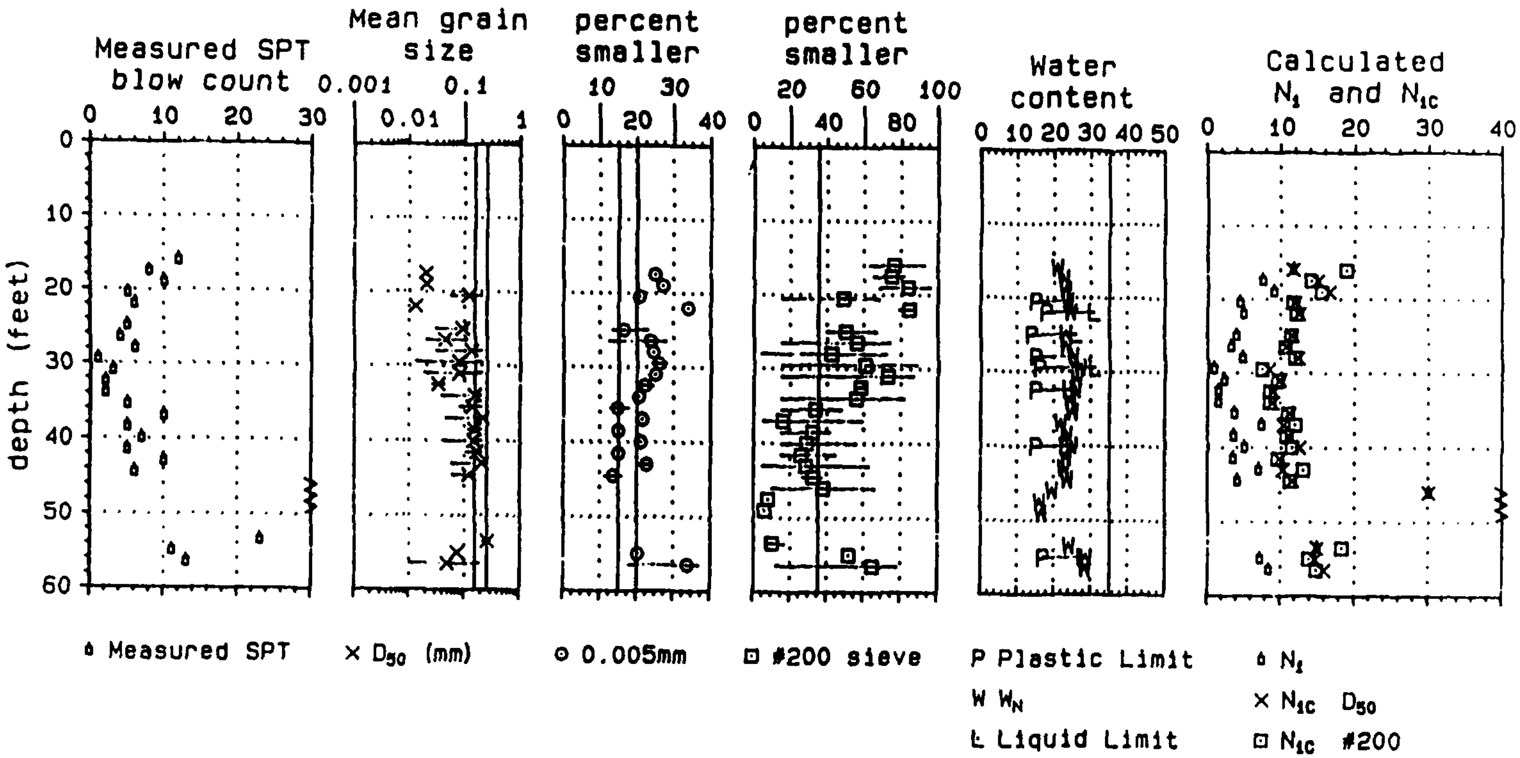
BEQ-26
Barkley dam

SPT Liquefaction Analysis



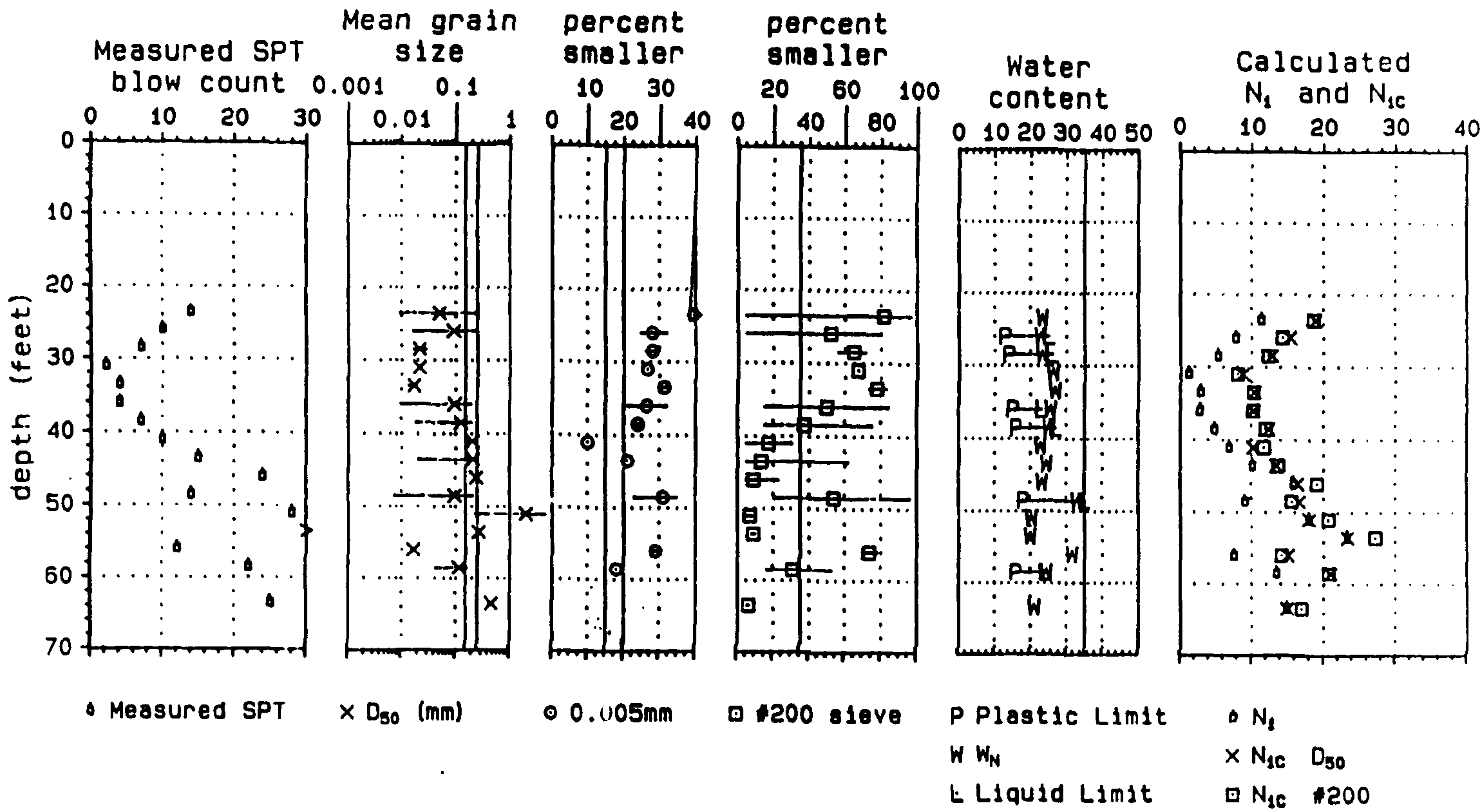
BEQ-27
Barkley dam

SPT Liquefaction Analysis



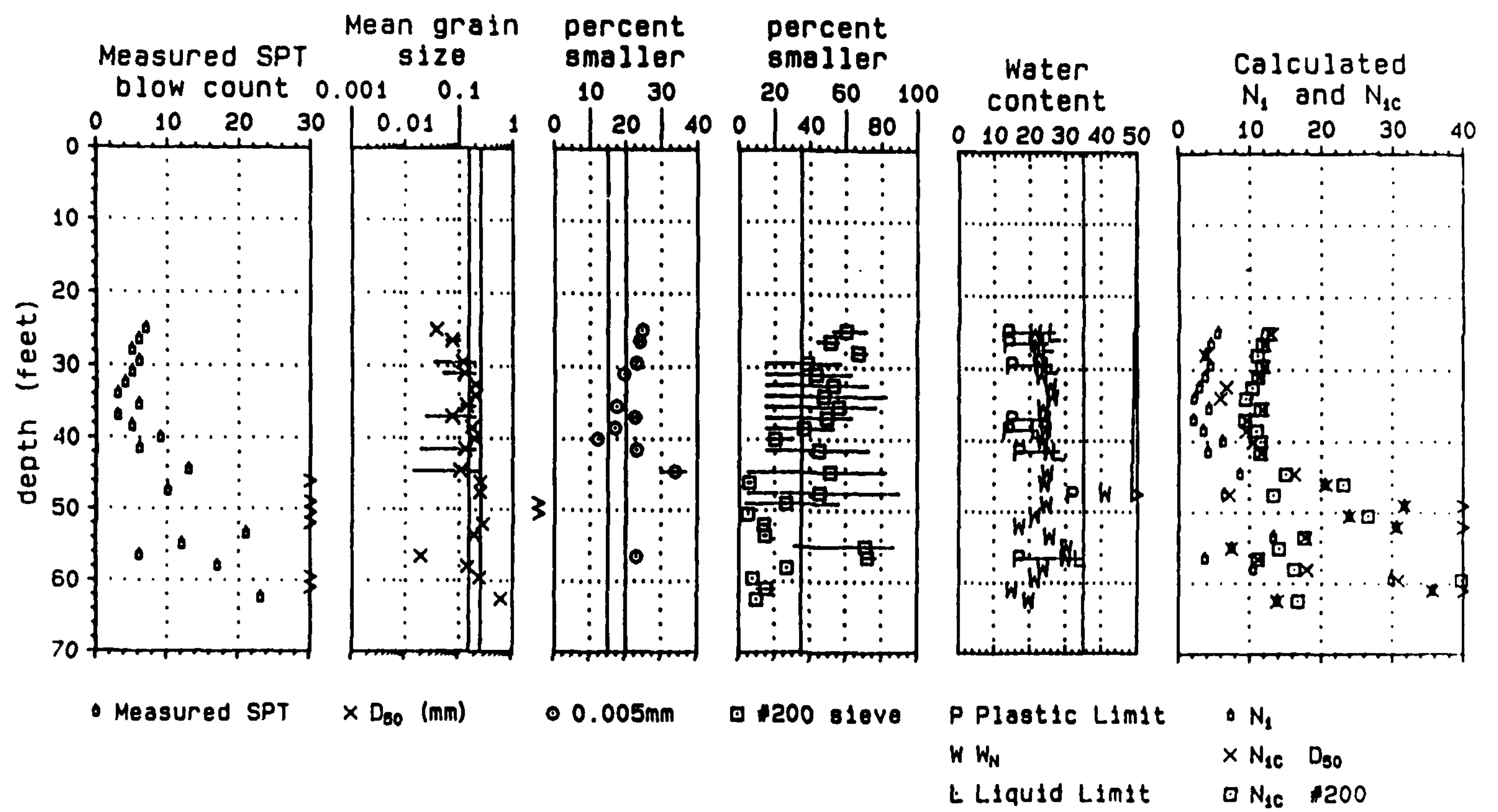
BEQ-28
Barkley dam

SPT Liquefaction Analysis



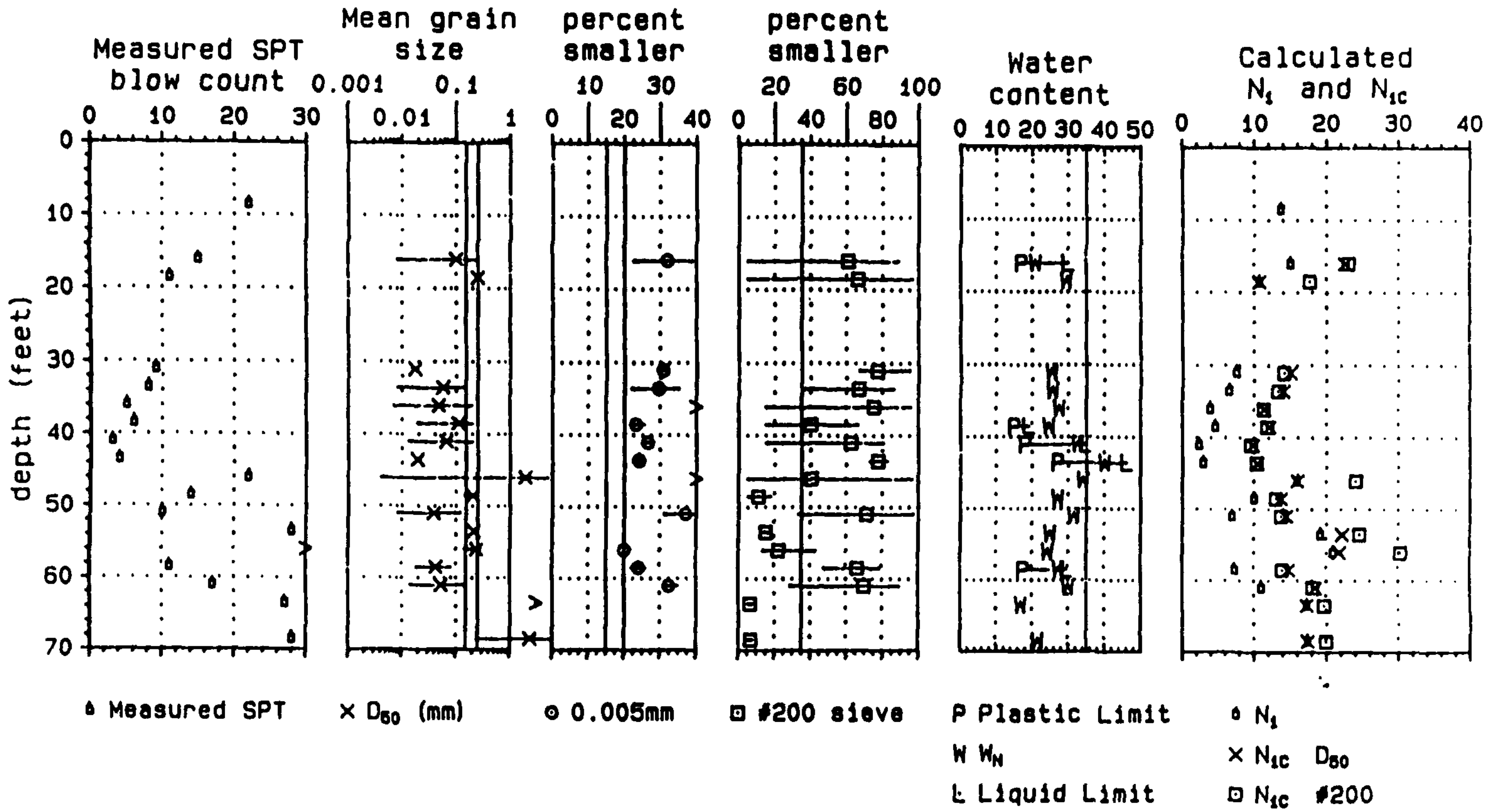
BEQ-29
Barkley dam

SPT Liquefaction Analysis



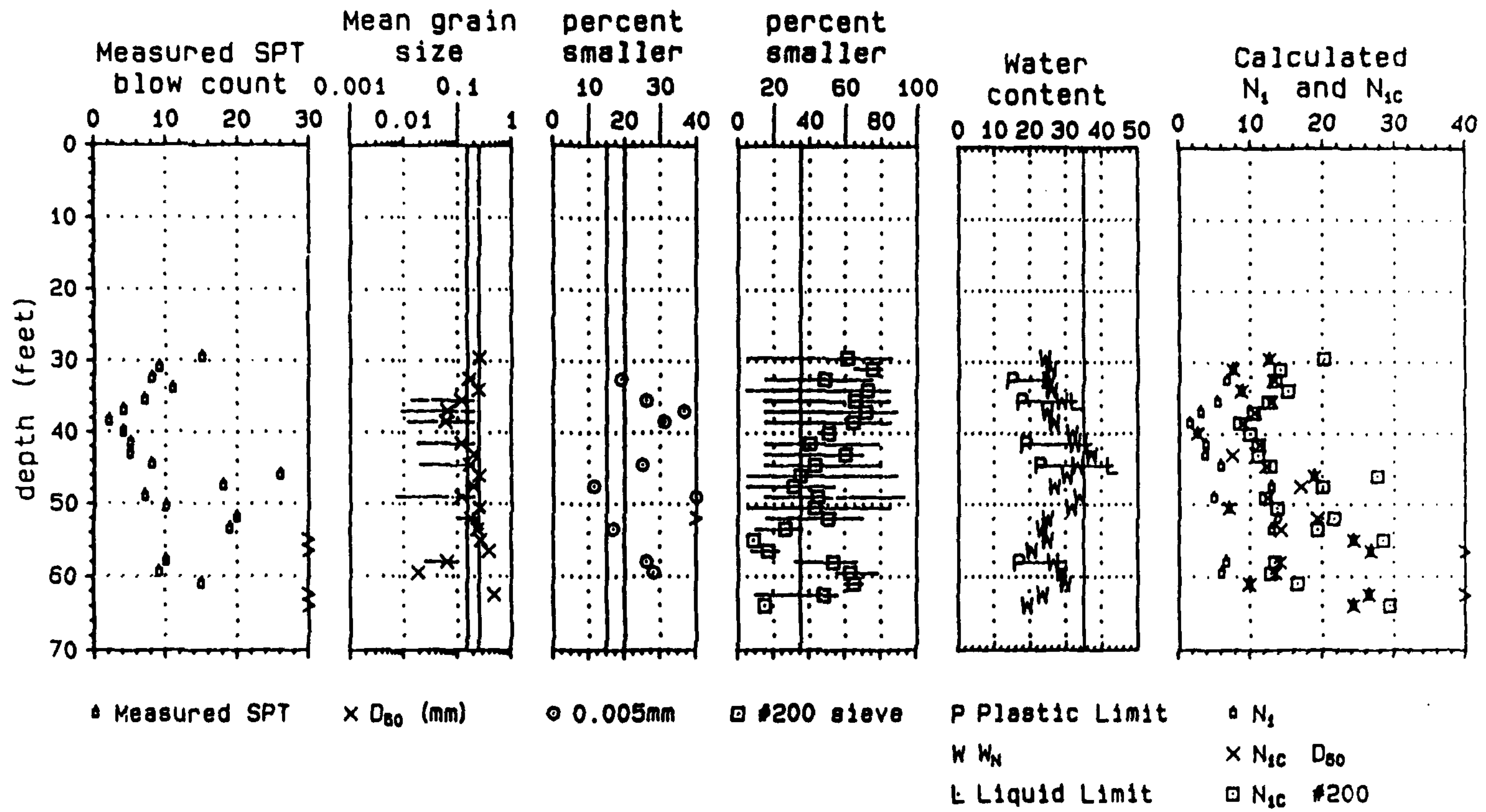
BEQ-30
Barkley dam

SPT Liquefaction Analysis



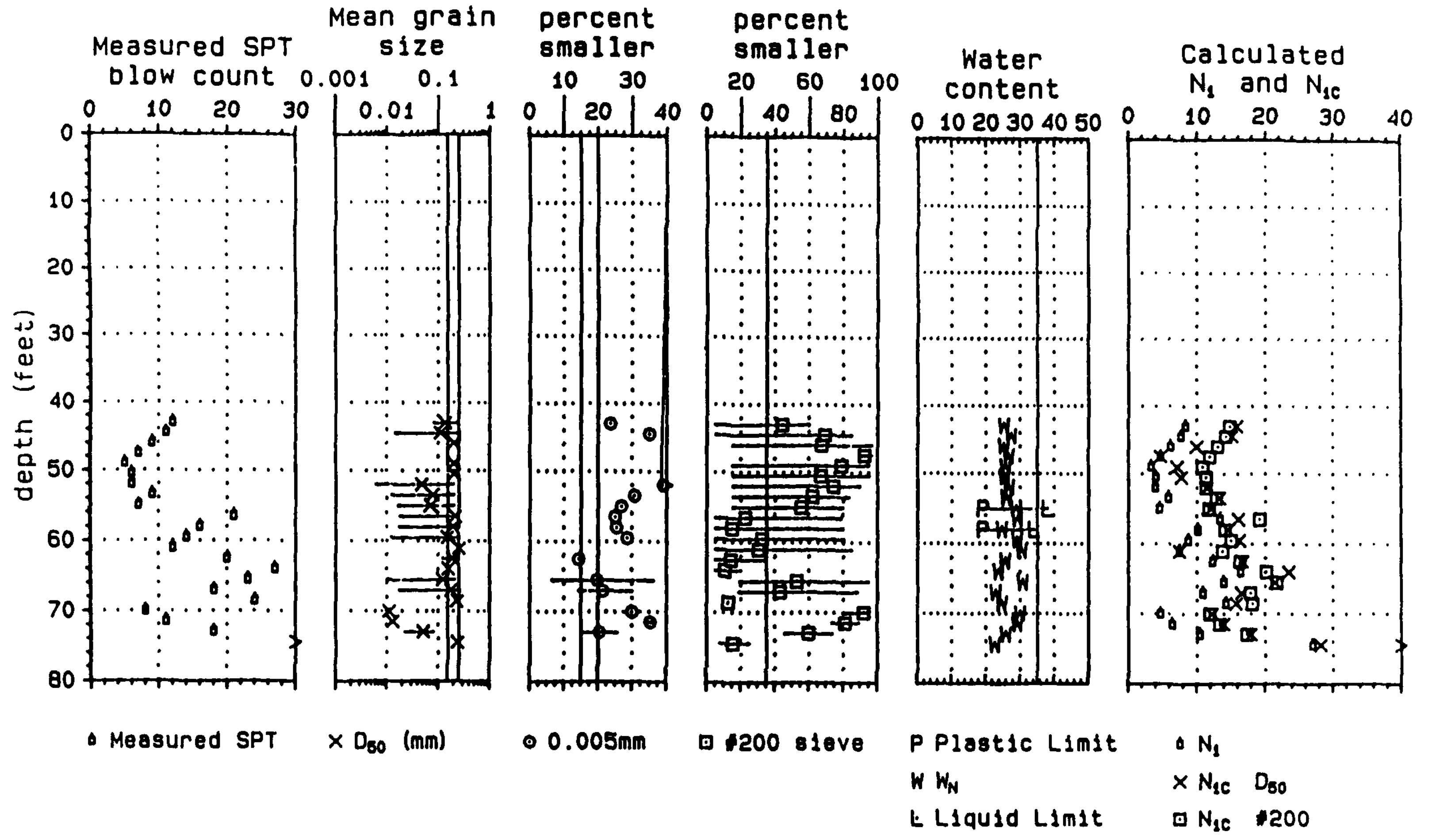
BEQ-31
Barkley dam

SPT Liquefaction Analysis



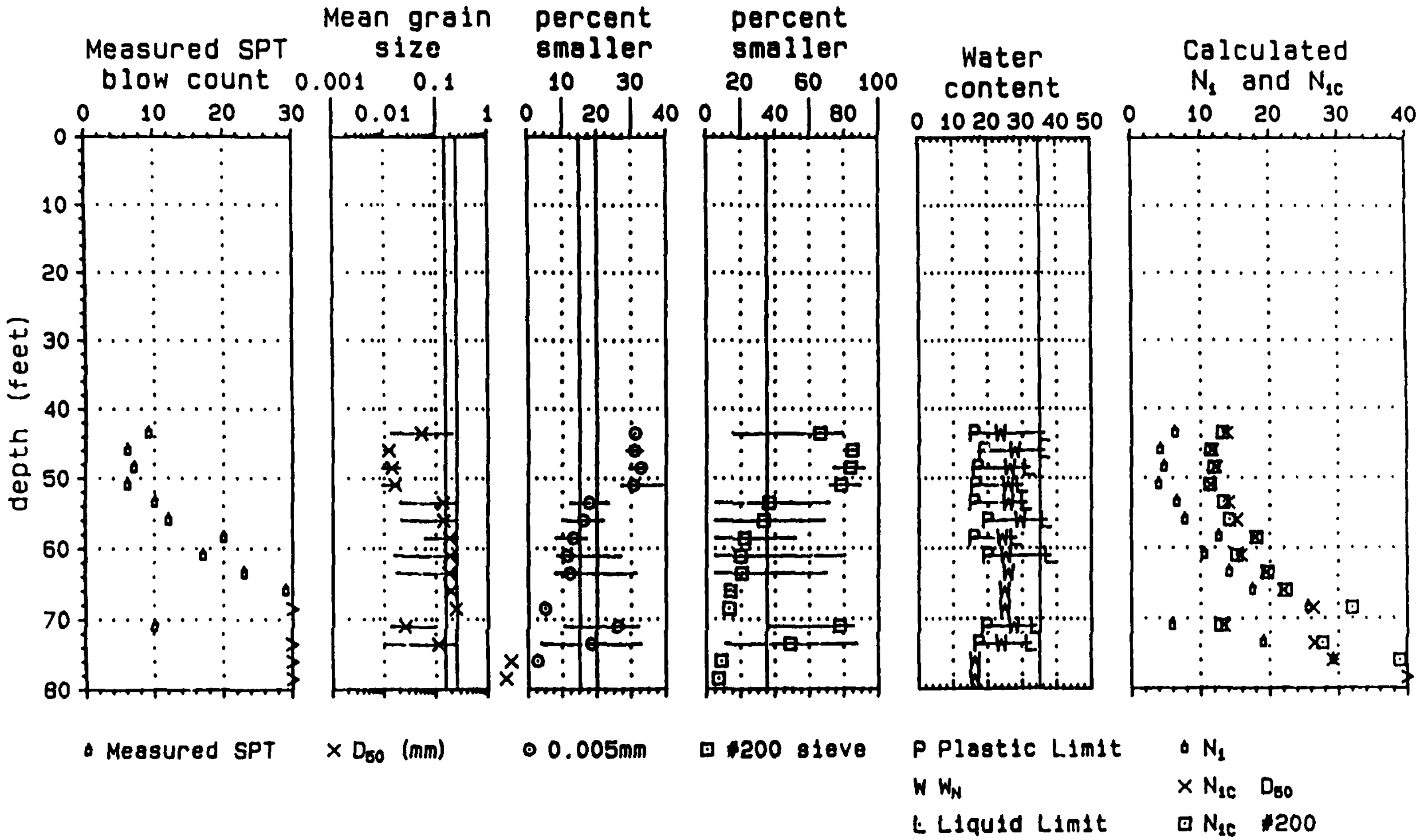
BEQ-32
Barkley dam

SPT Liquefaction Analysis



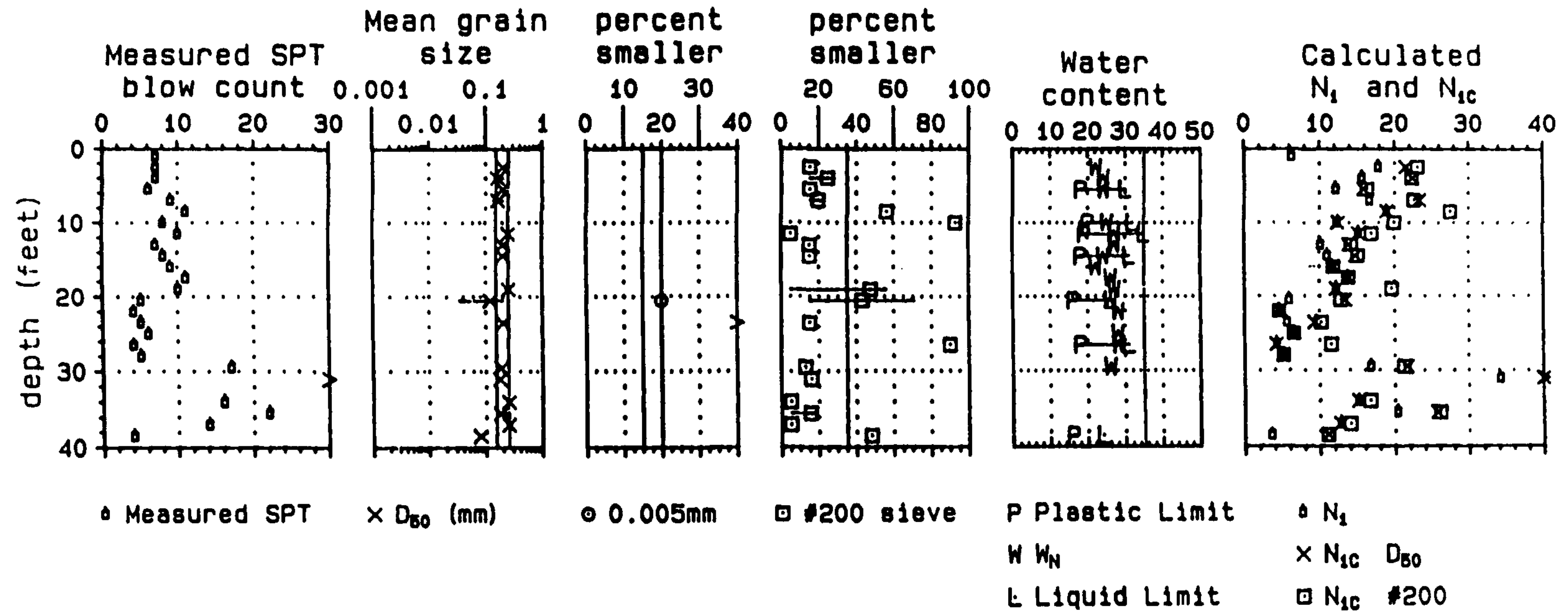
BEQ-33
Barkley dam

SPT Liquefaction Analysis



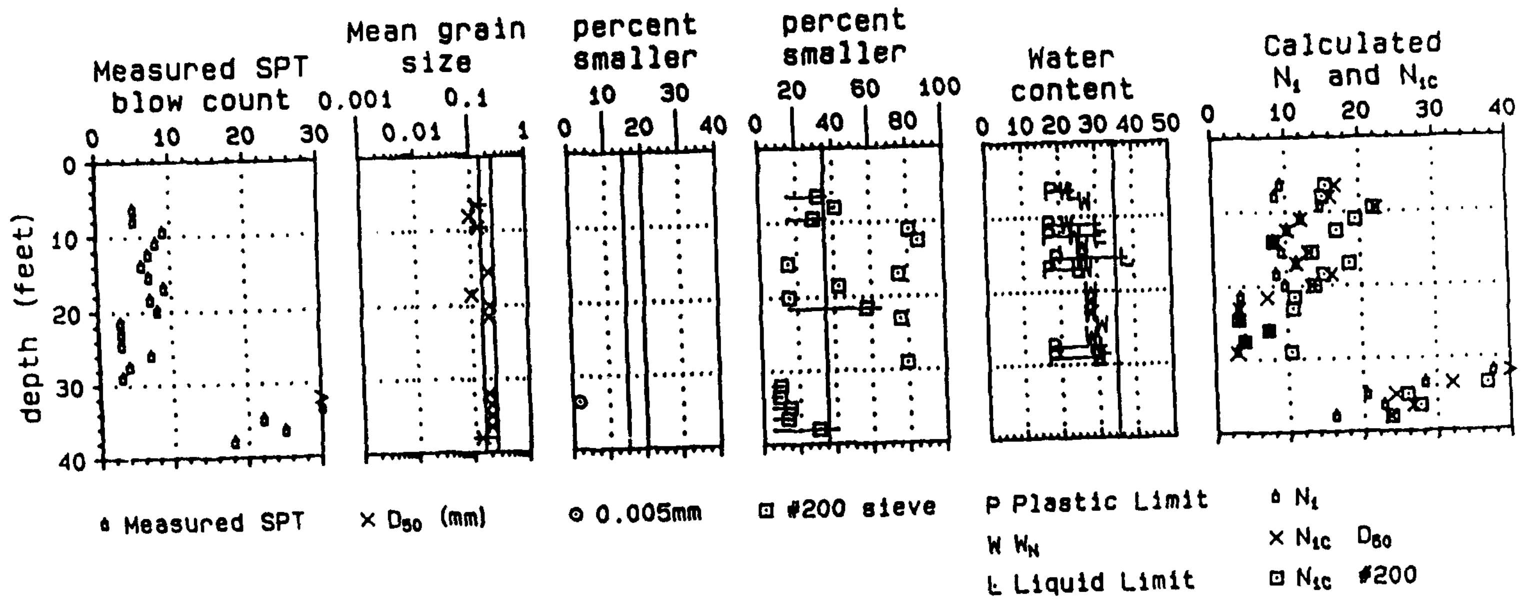
BEQ-34
Barkley dam

SPT Liquefaction Analysis



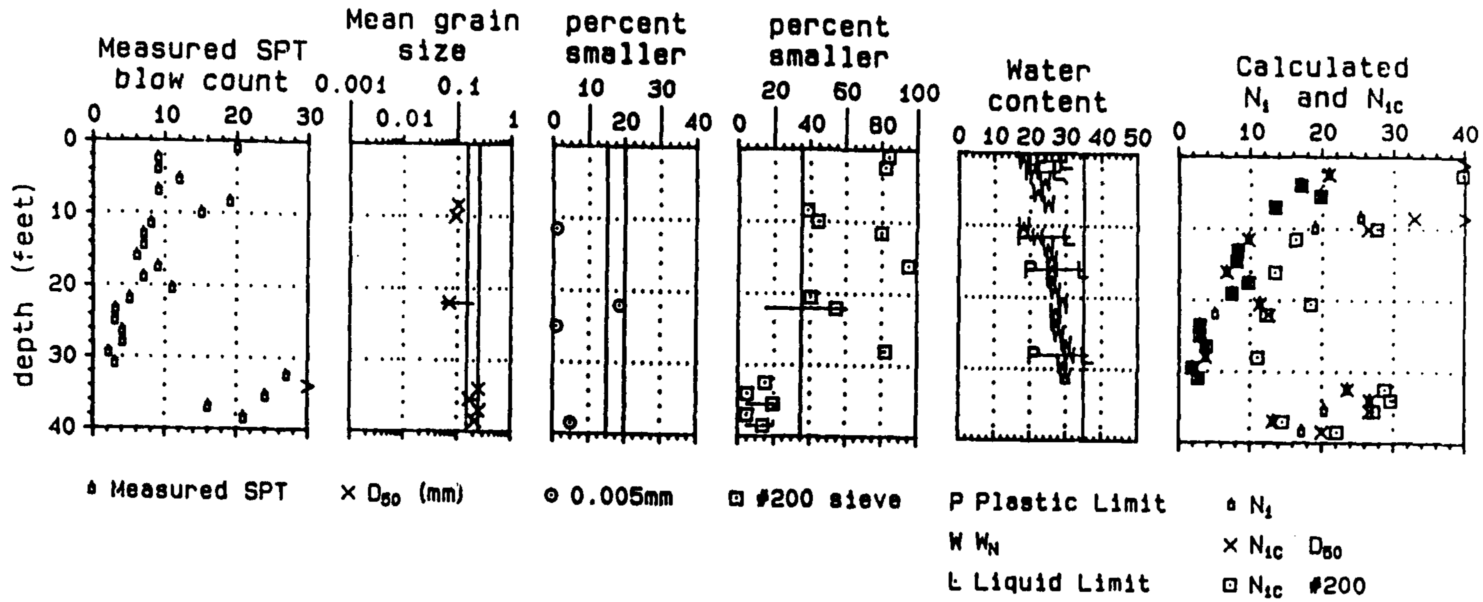
B-D-1
Barkley dam

SPT Liquefaction Analysis



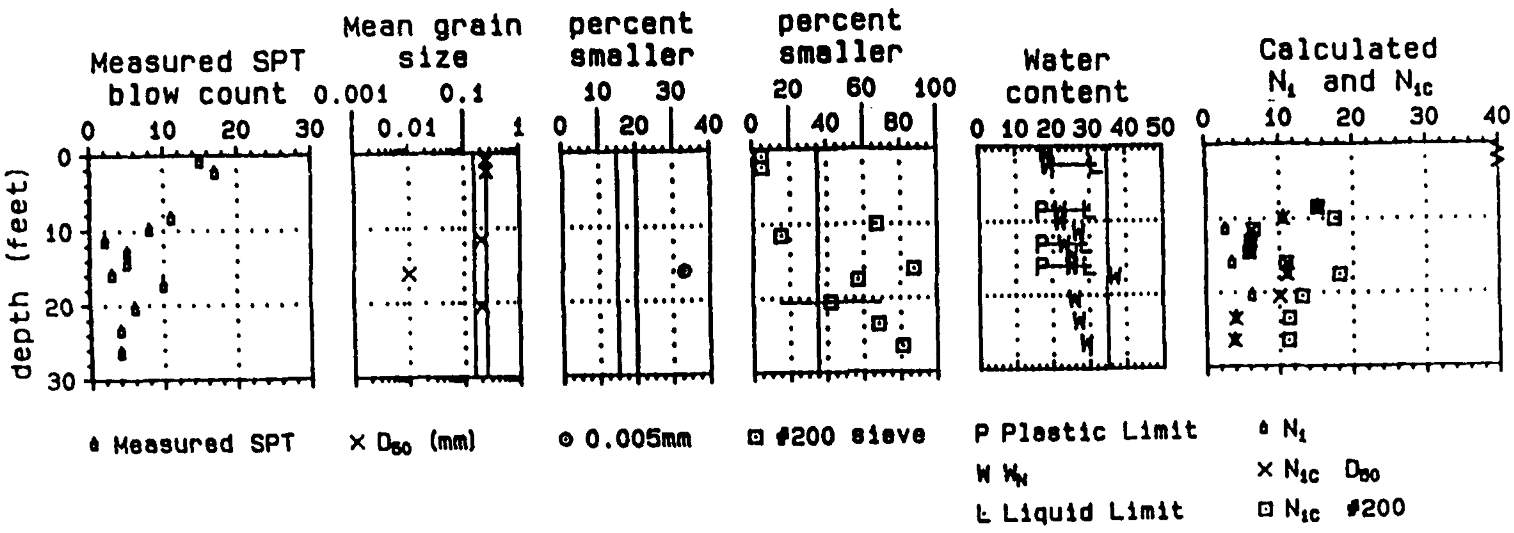
B-D-2
Barkley dam

SPT Liquefaction Analysis



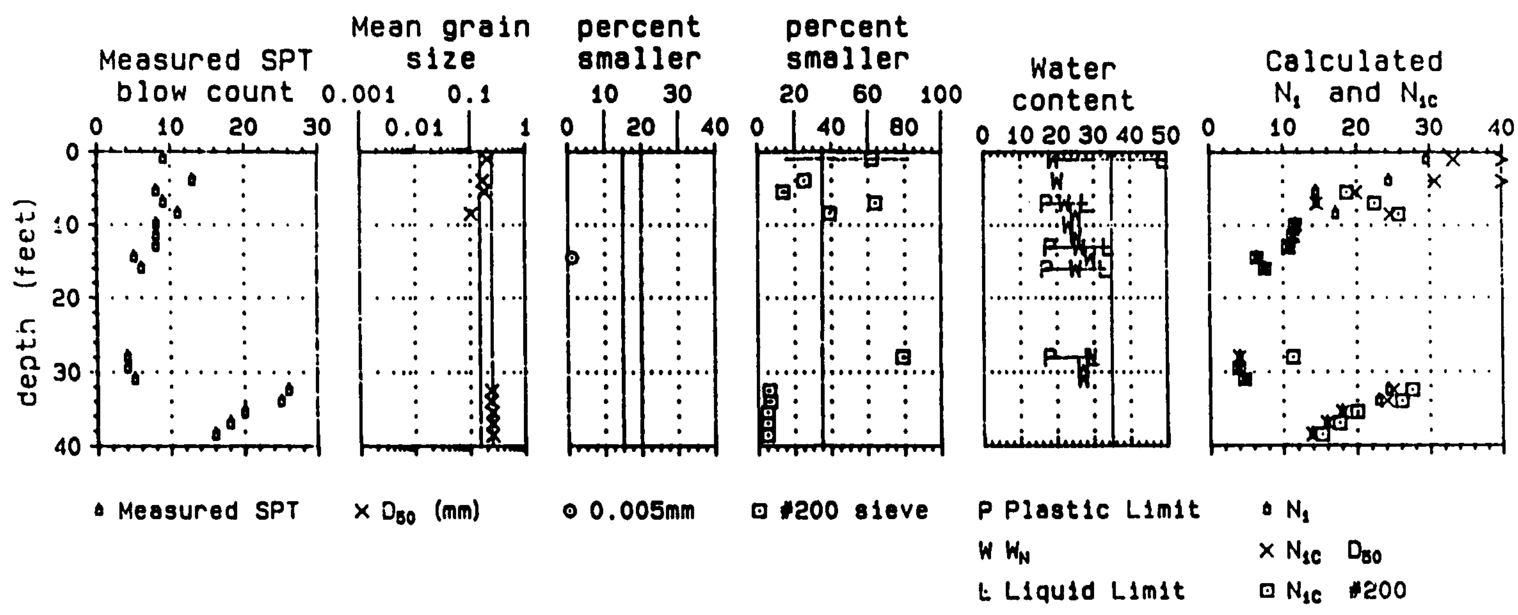
B-D-3
Barkley dam

SPT Liquefaction Analysis



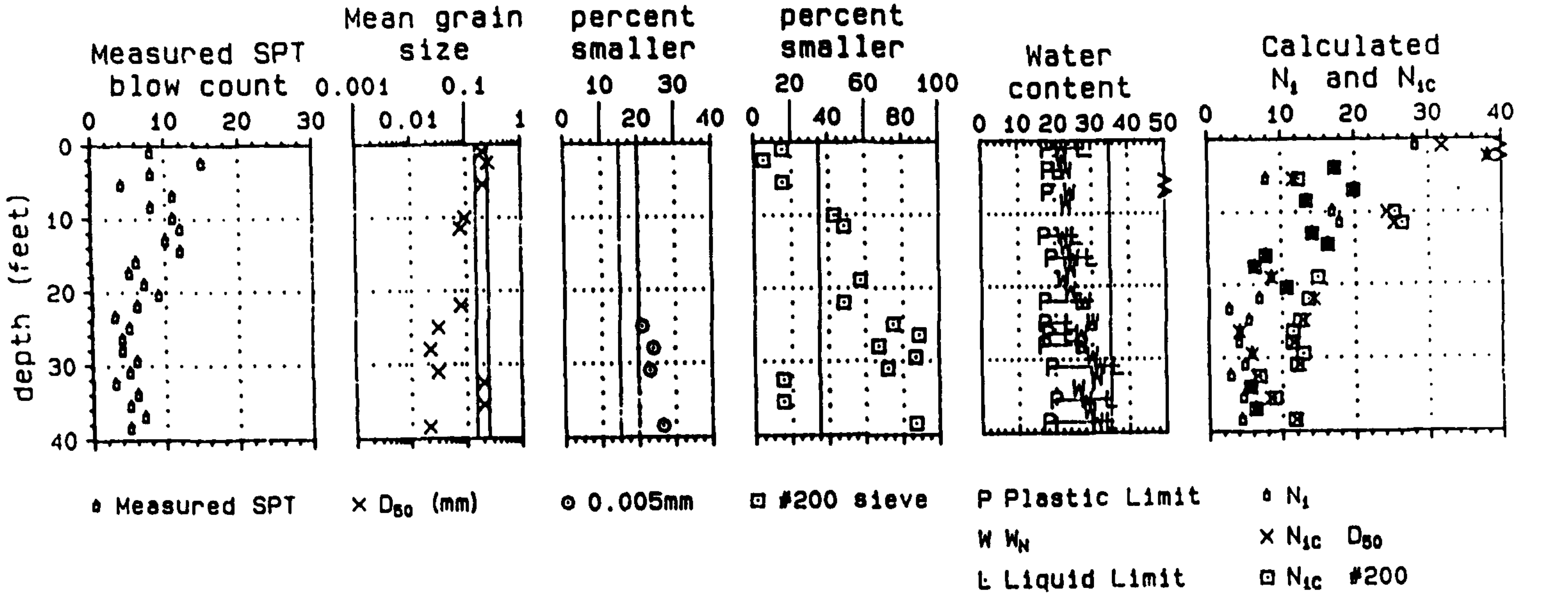
B-D-4
Barkley dam

SPT Liquefaction Analysis



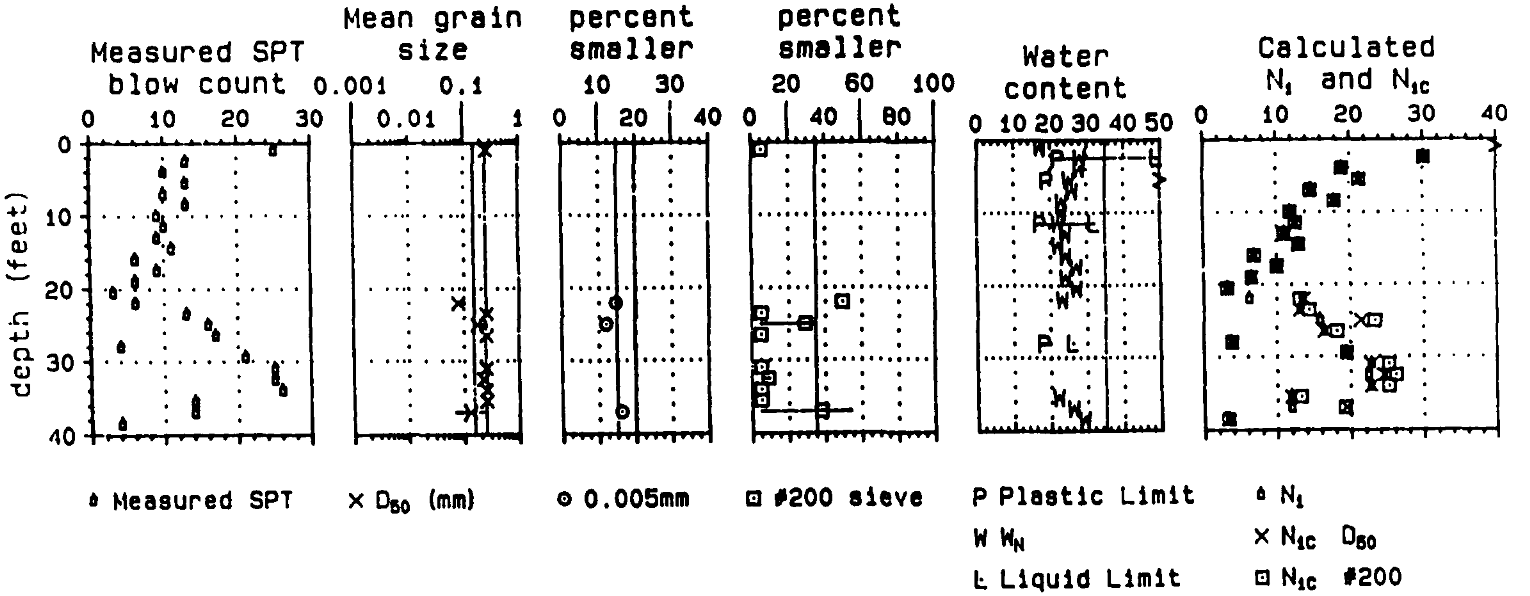
B-D-5
Barkley dam

SPT Liquefaction Analysis



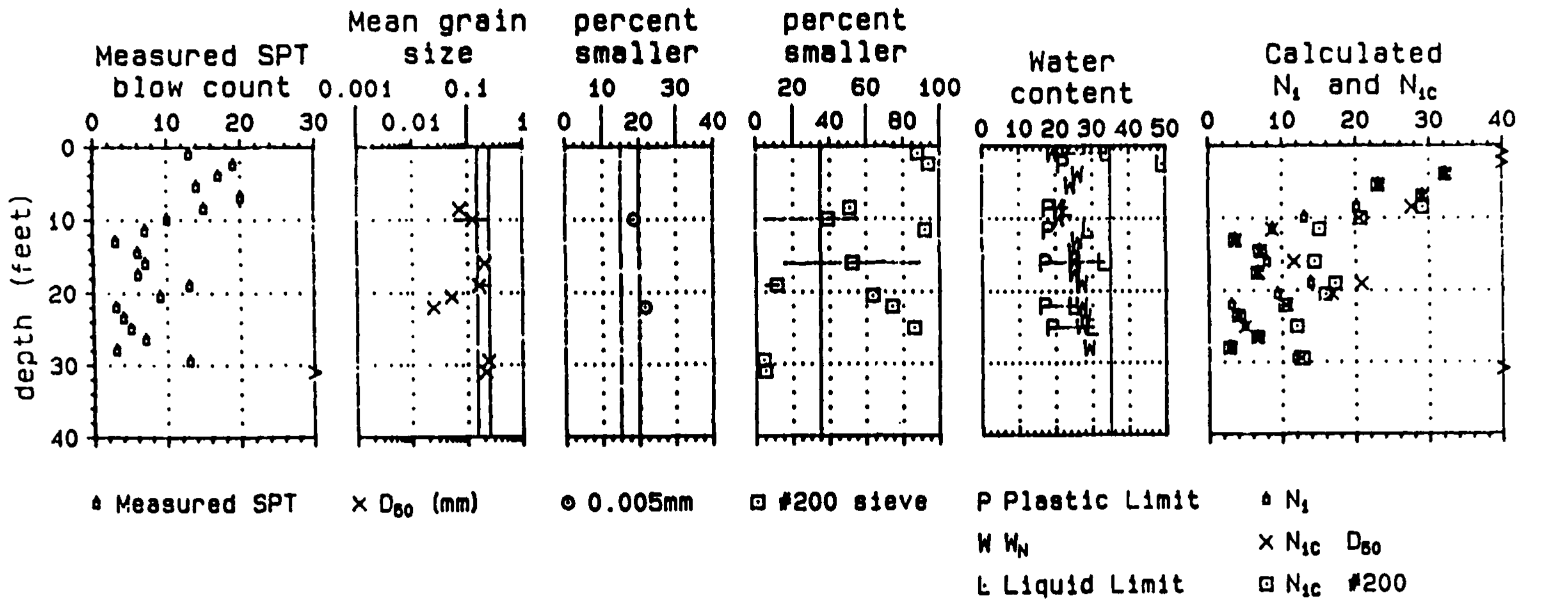
B-D-6
Barkley dam

SPT Liquefaction Analysis



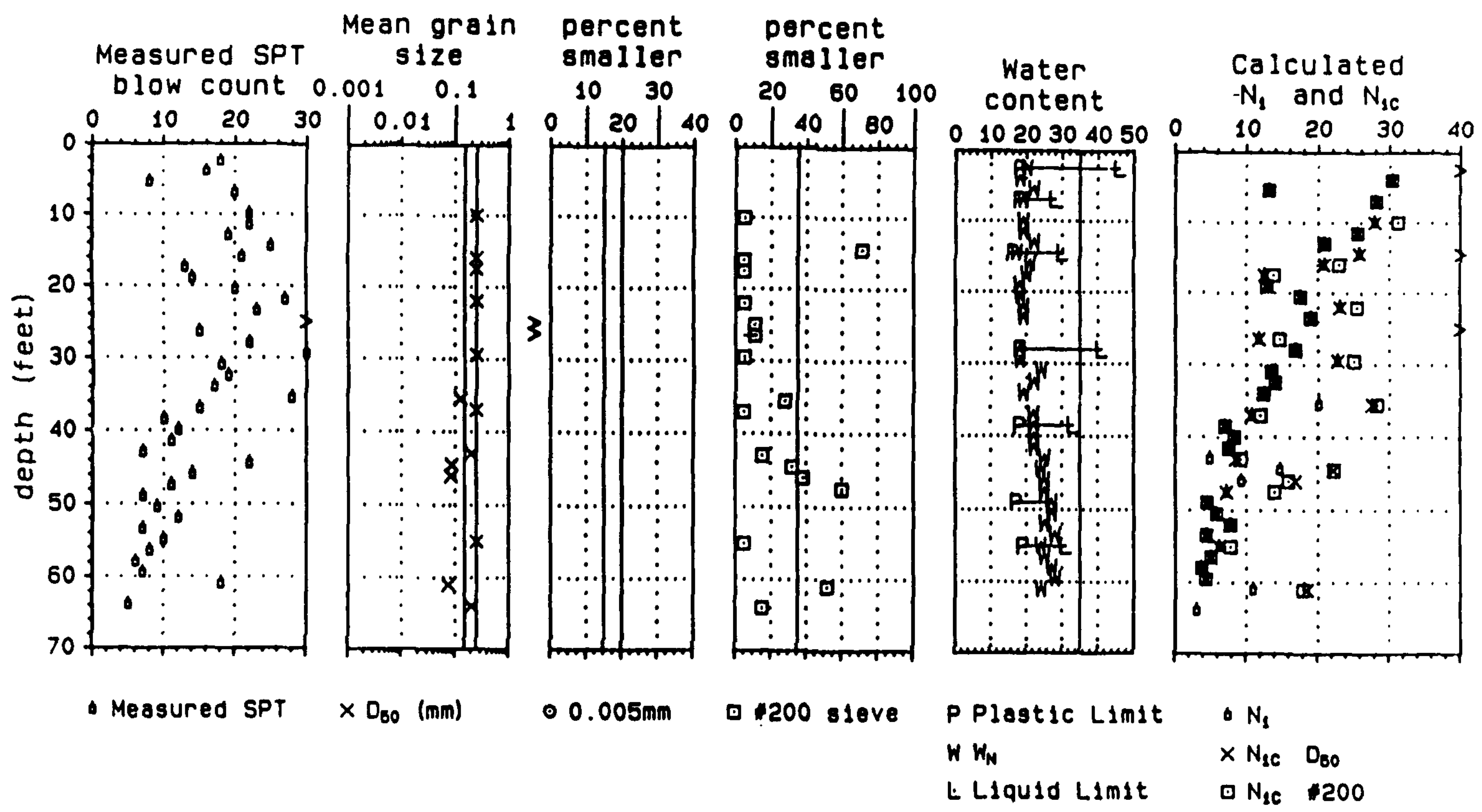
B-D-7
Barkley dam

SPT Liquefaction Analysis



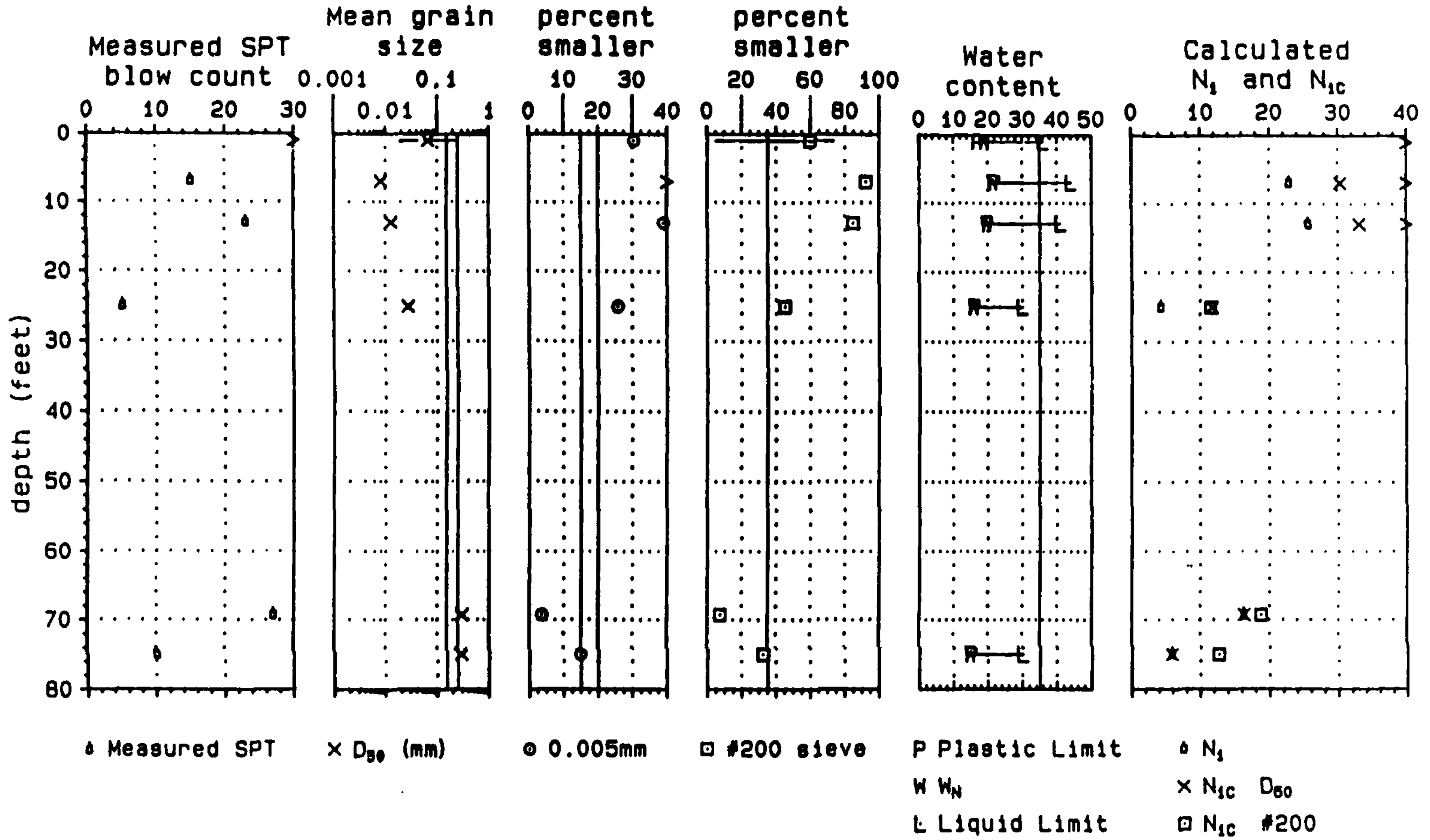
B-D-8
Barkley dam

SPT Liquefaction Analysis



B-D-9
Barkley dam

SPT Liquefaction Analysis

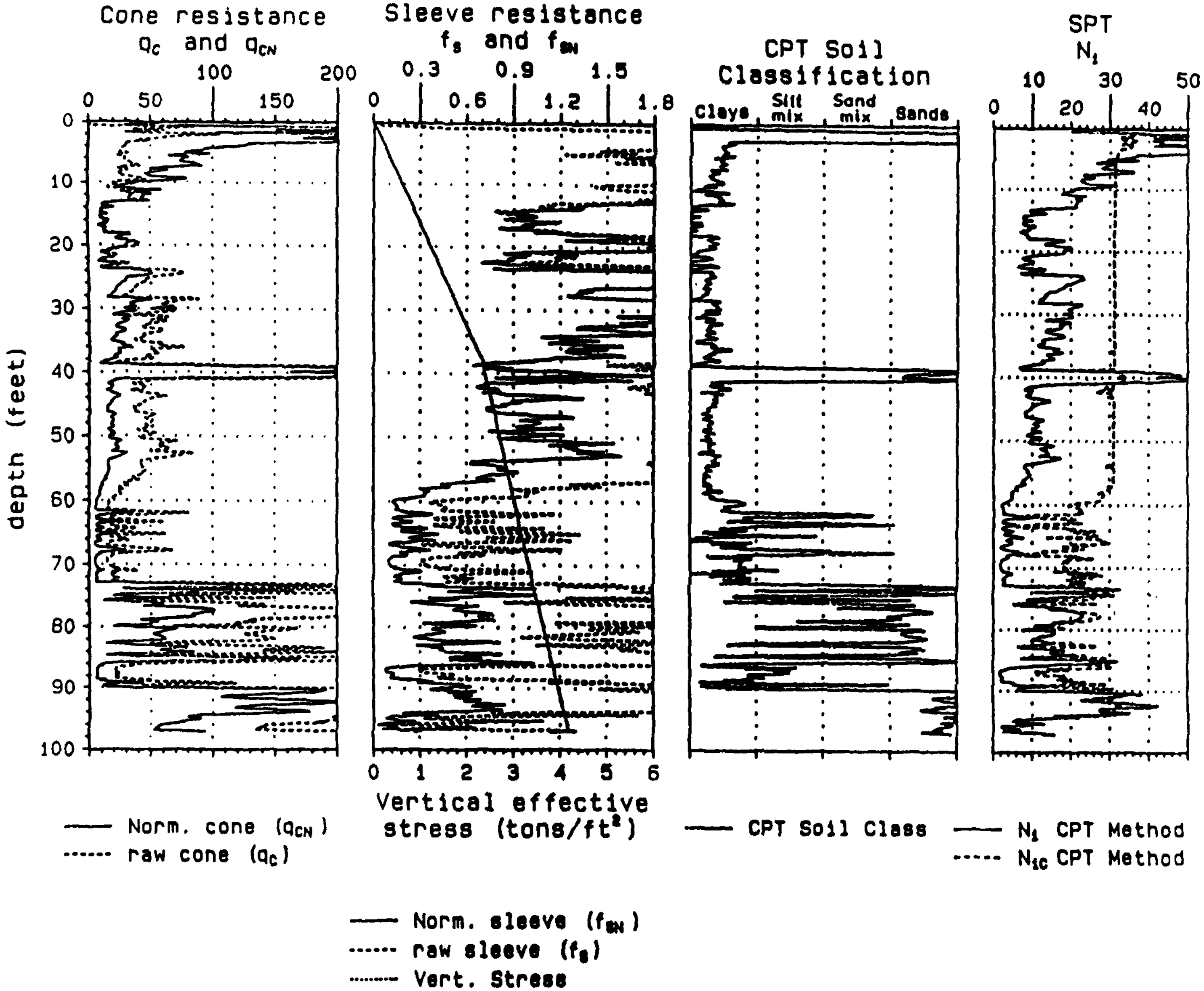


DS-3
Barkley dam

SPT Liquefaction Analysis

APPENDIX B:

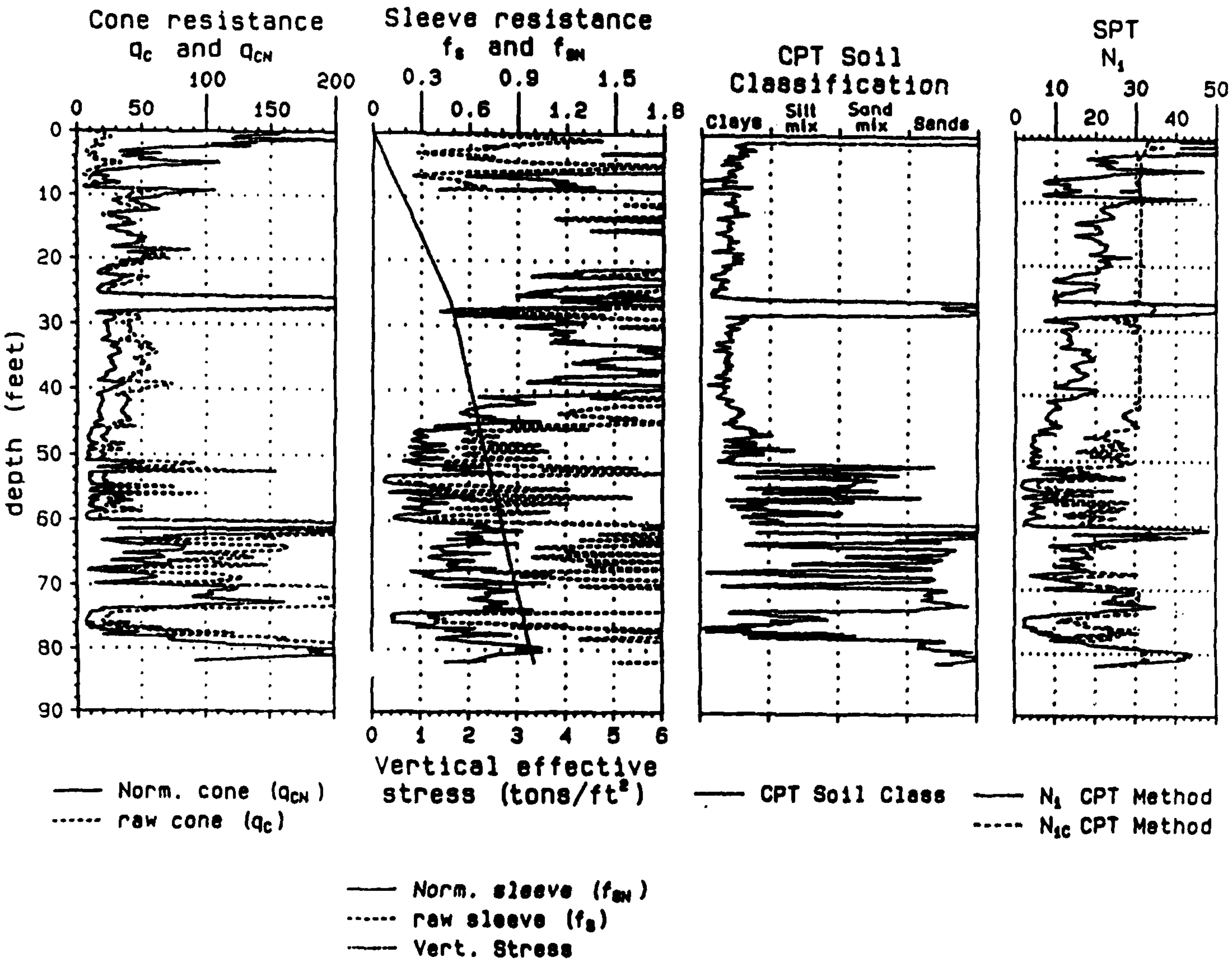
CPT DATA PLOTS



CPT Analysis

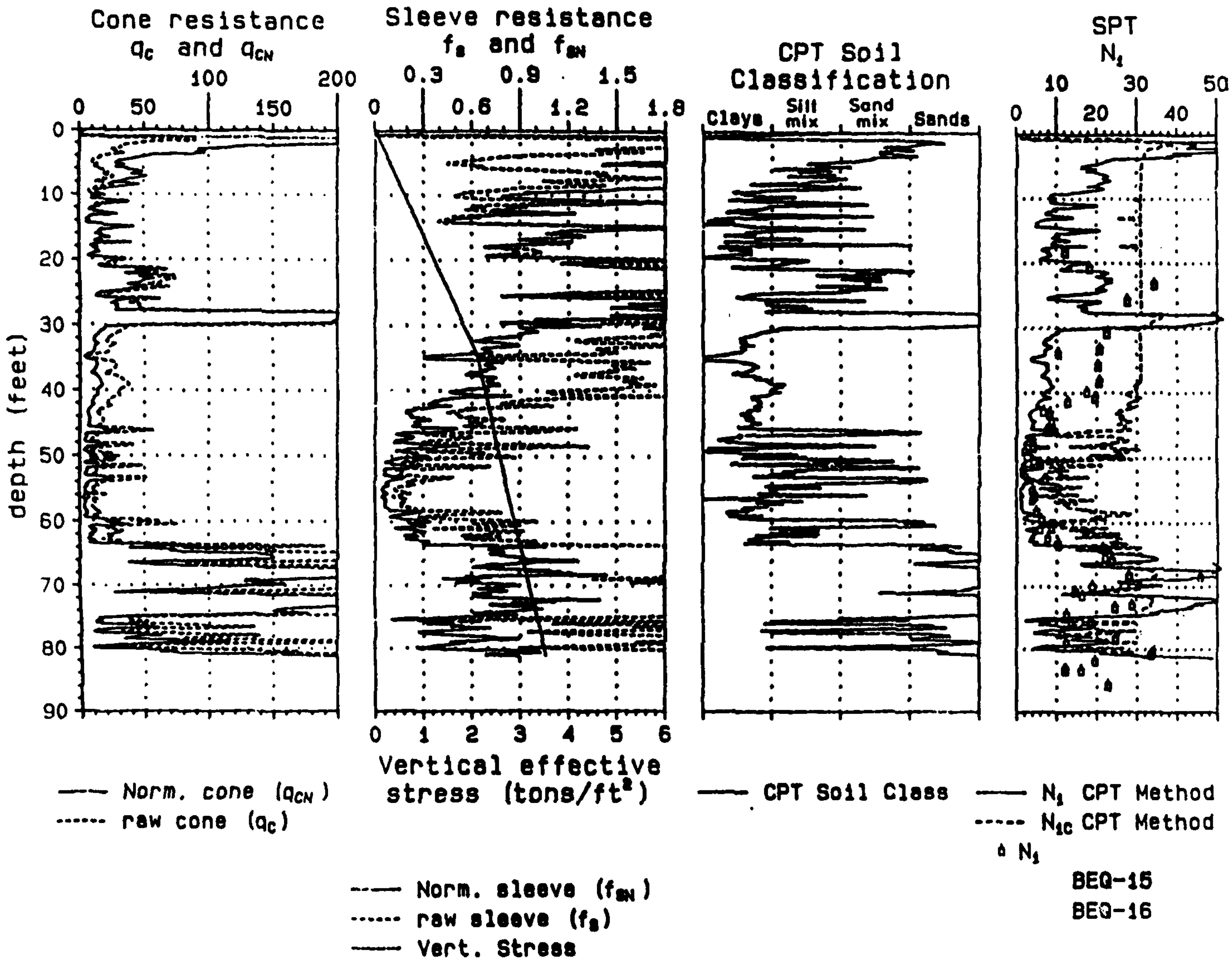
CPT-1

BARKLEY DAM



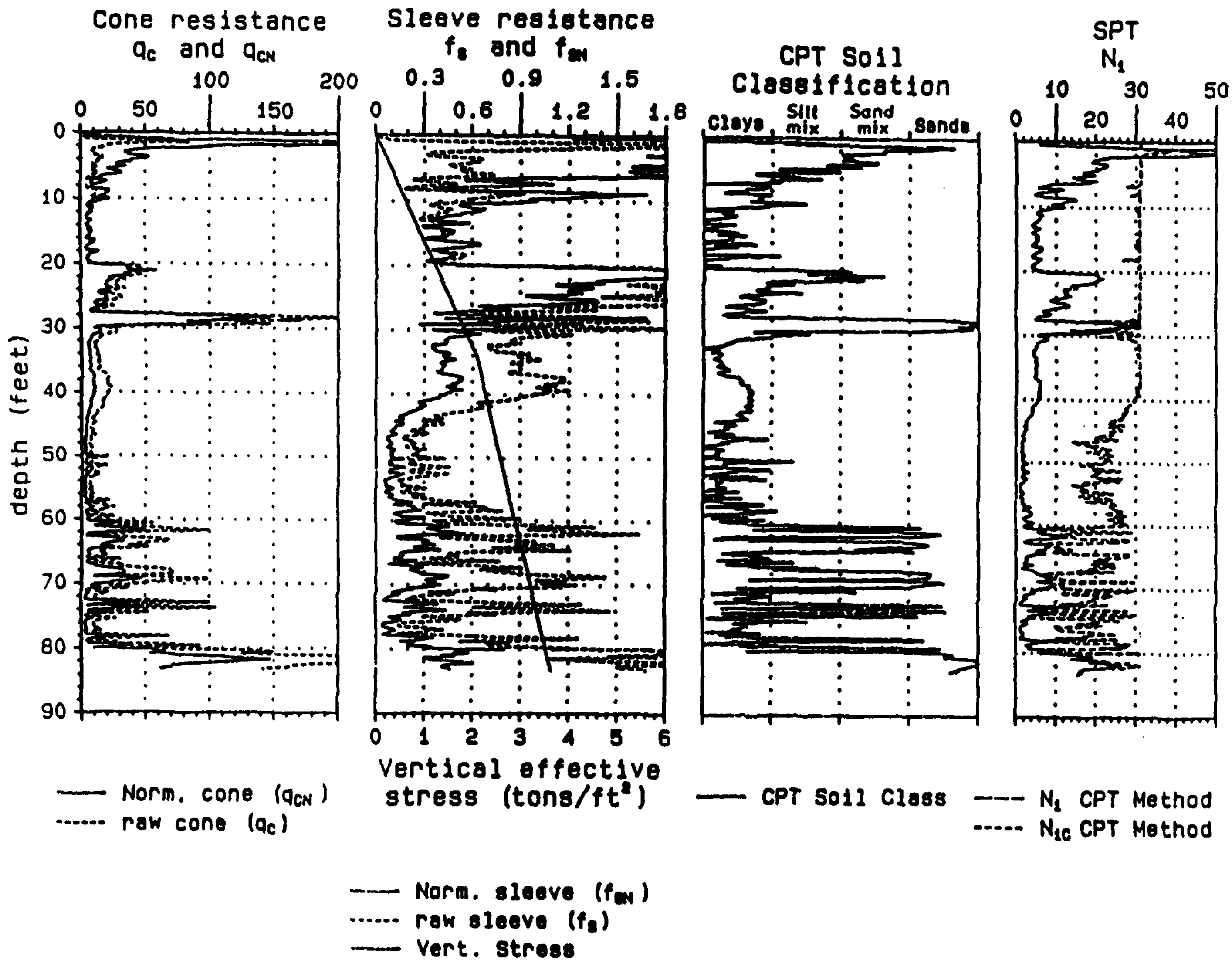
CPT Analysis

CPT-2
BARKLEY DAM



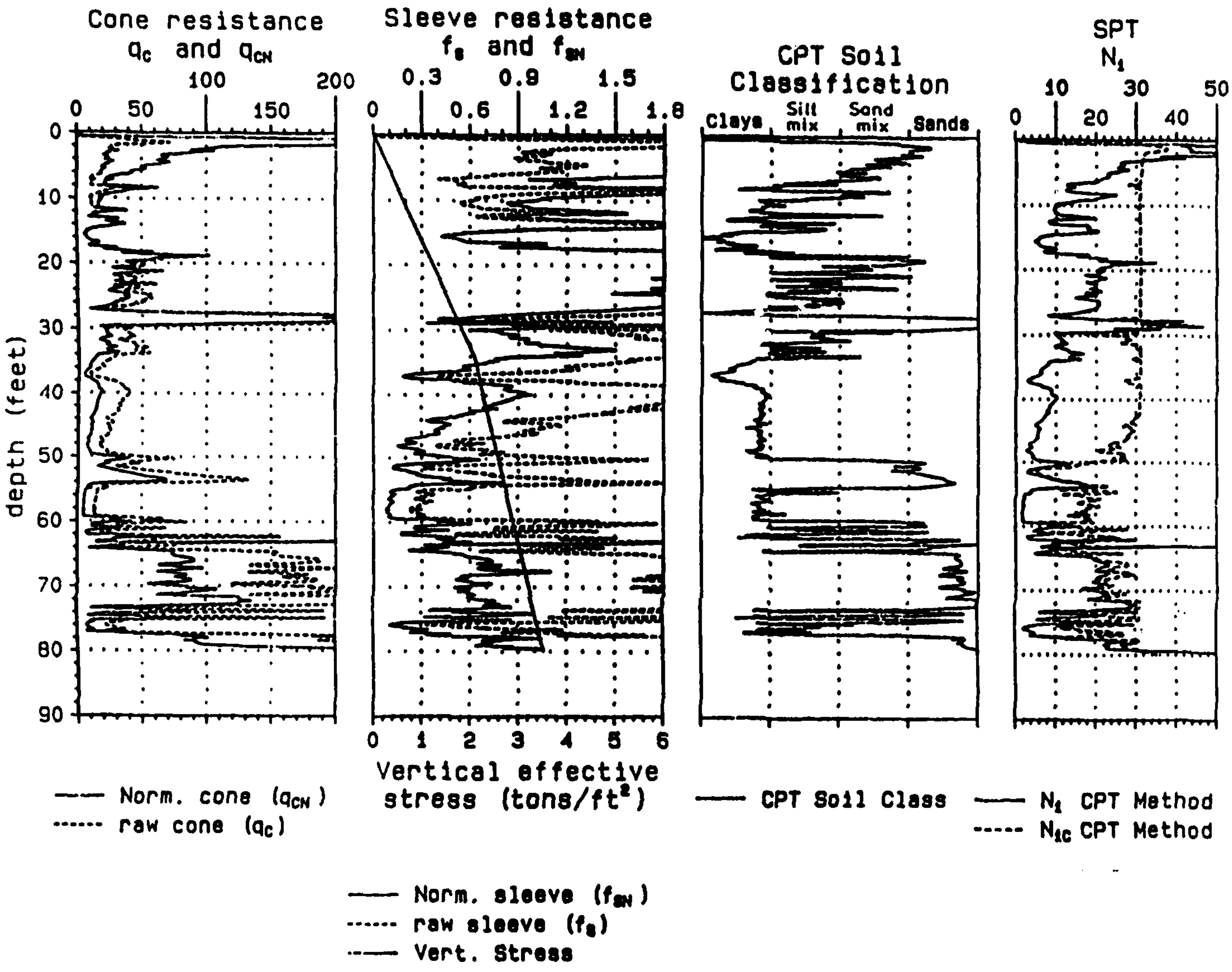
CPT Analysis

CPT-3
BARKLEY DAM



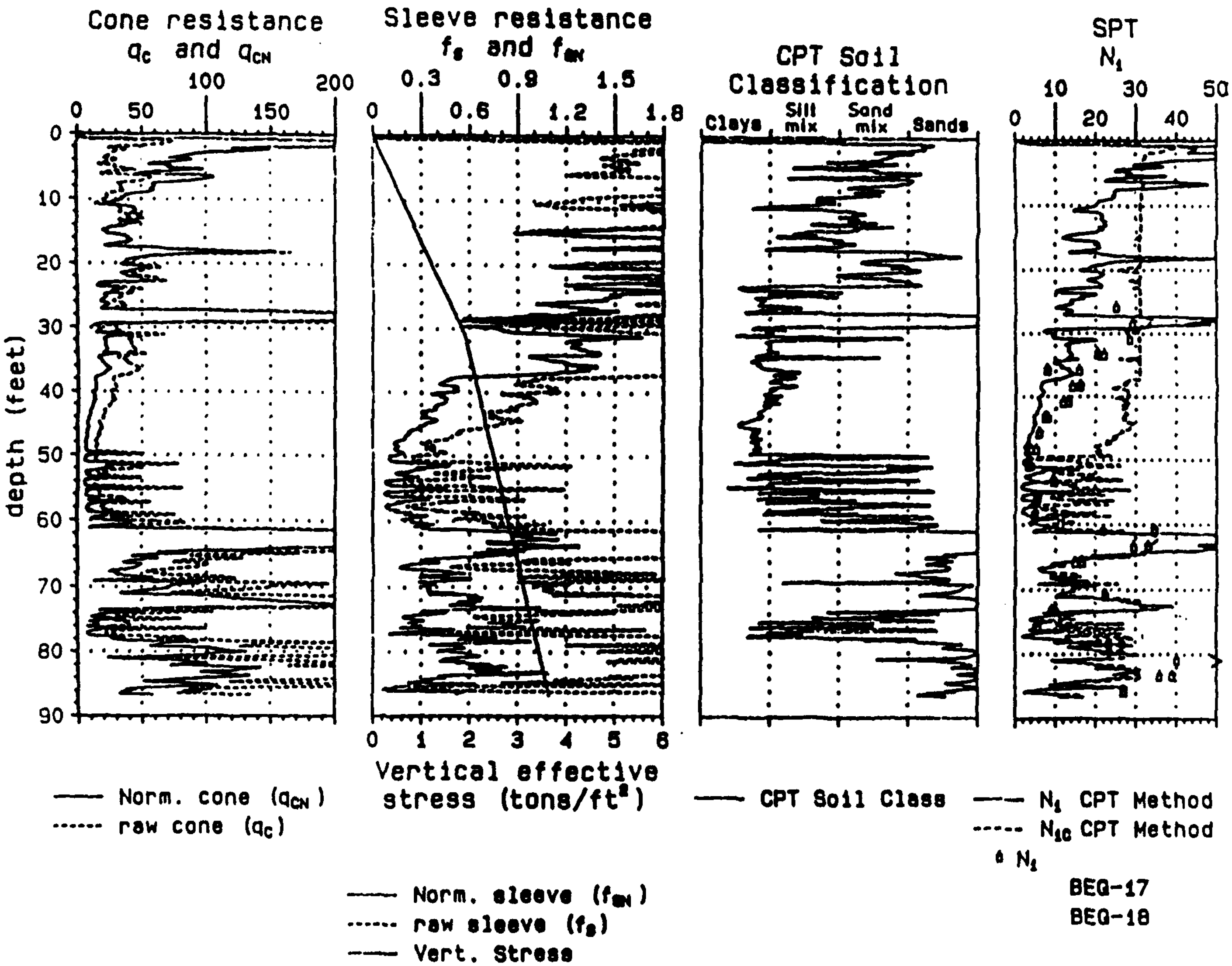
CPT Analysis

CPT-4
BARKLEY DAM



CPT Analysis

CPT-5
BARKLEY DAM

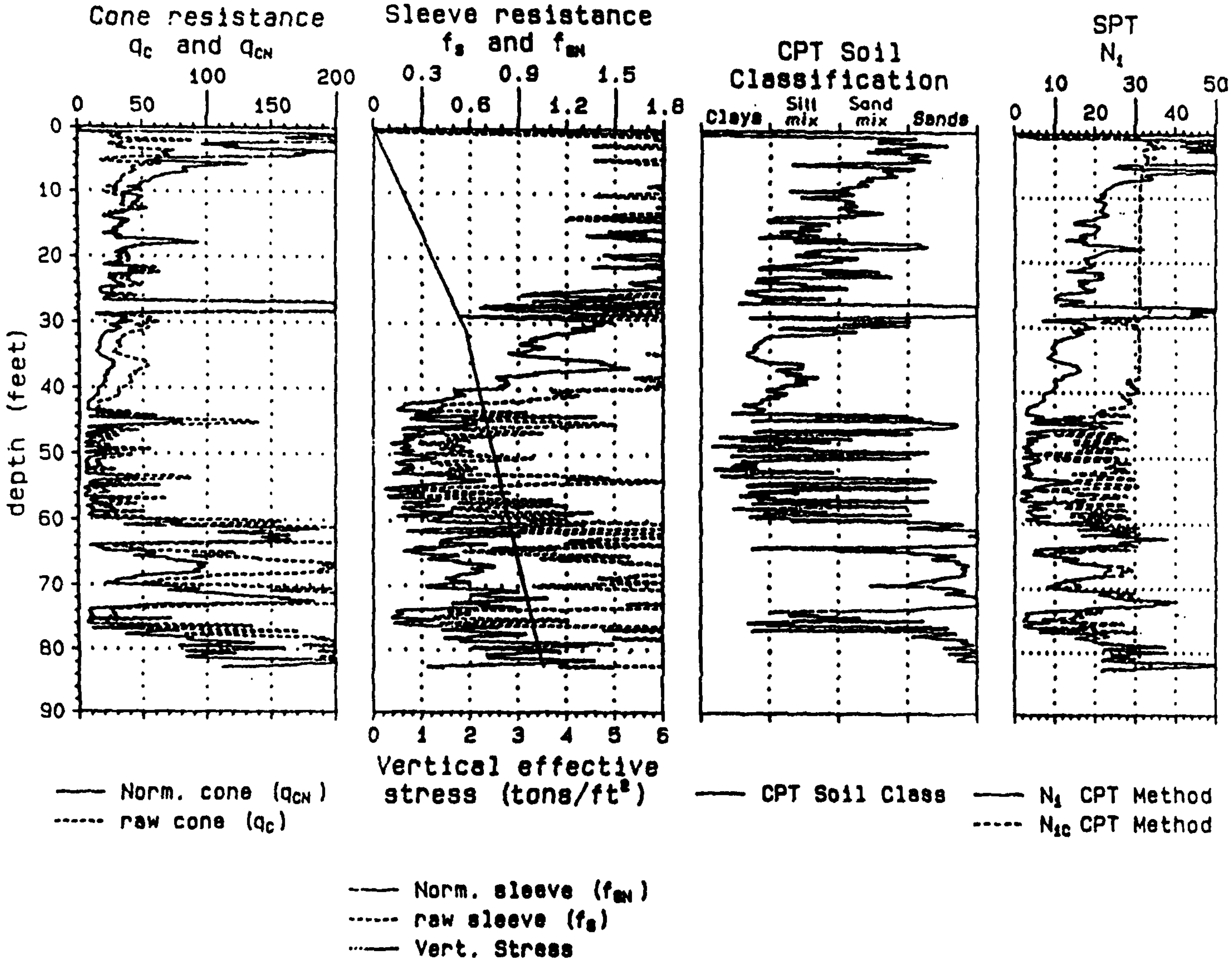


B7

CPT Analysis

CPT-6

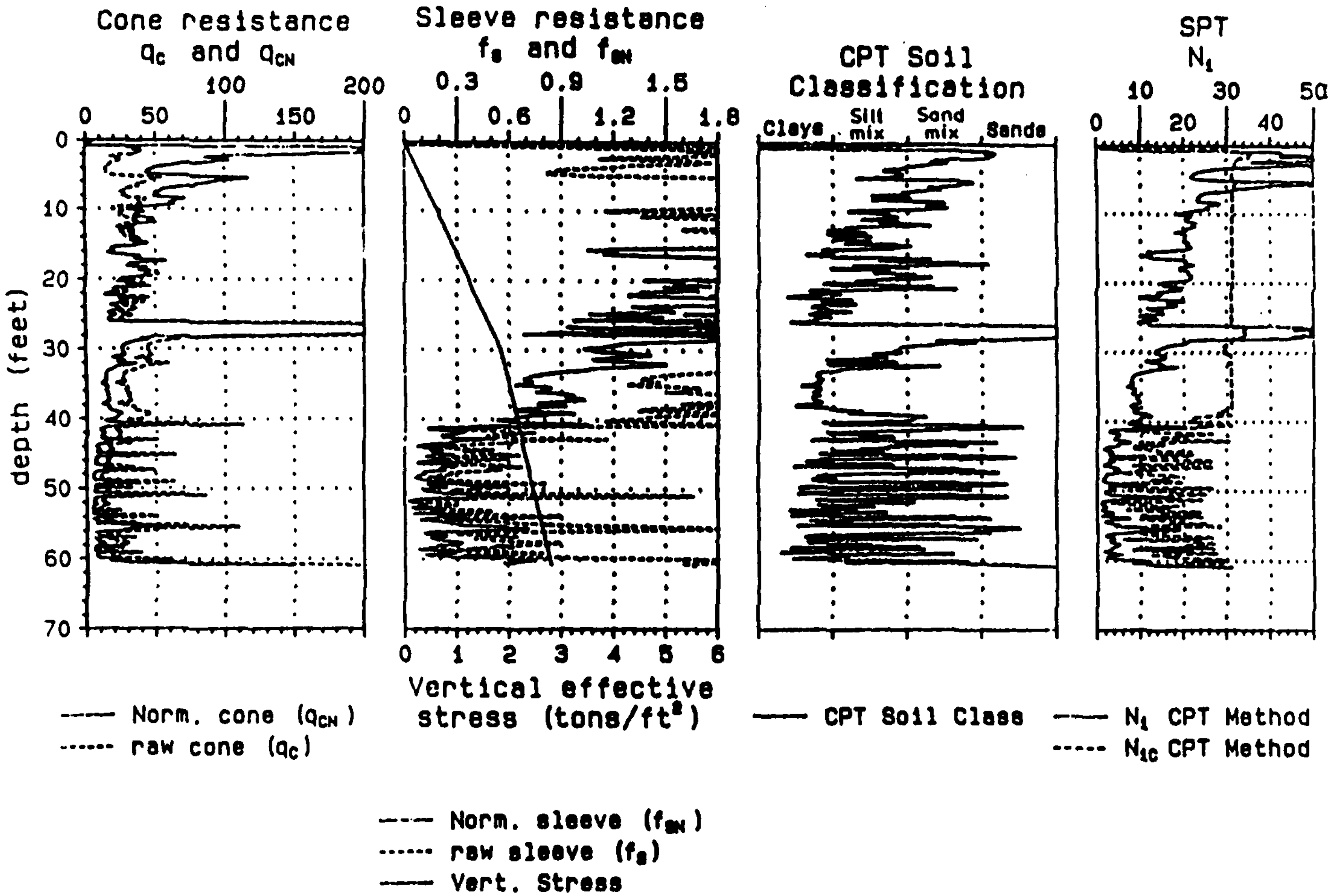
BARKLEY DAM



CPT Analysis

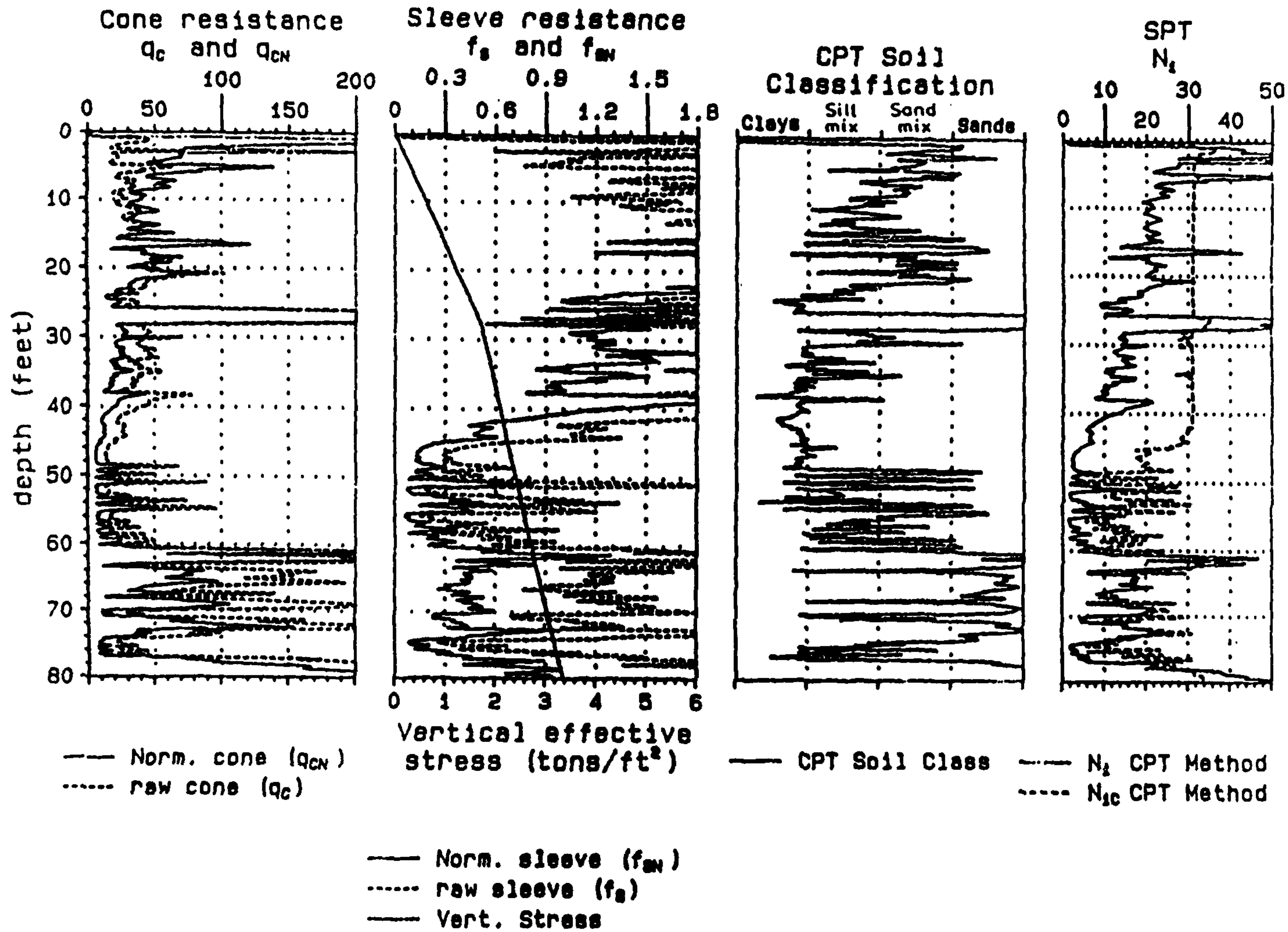
CPT-7

BARKLEY DAM



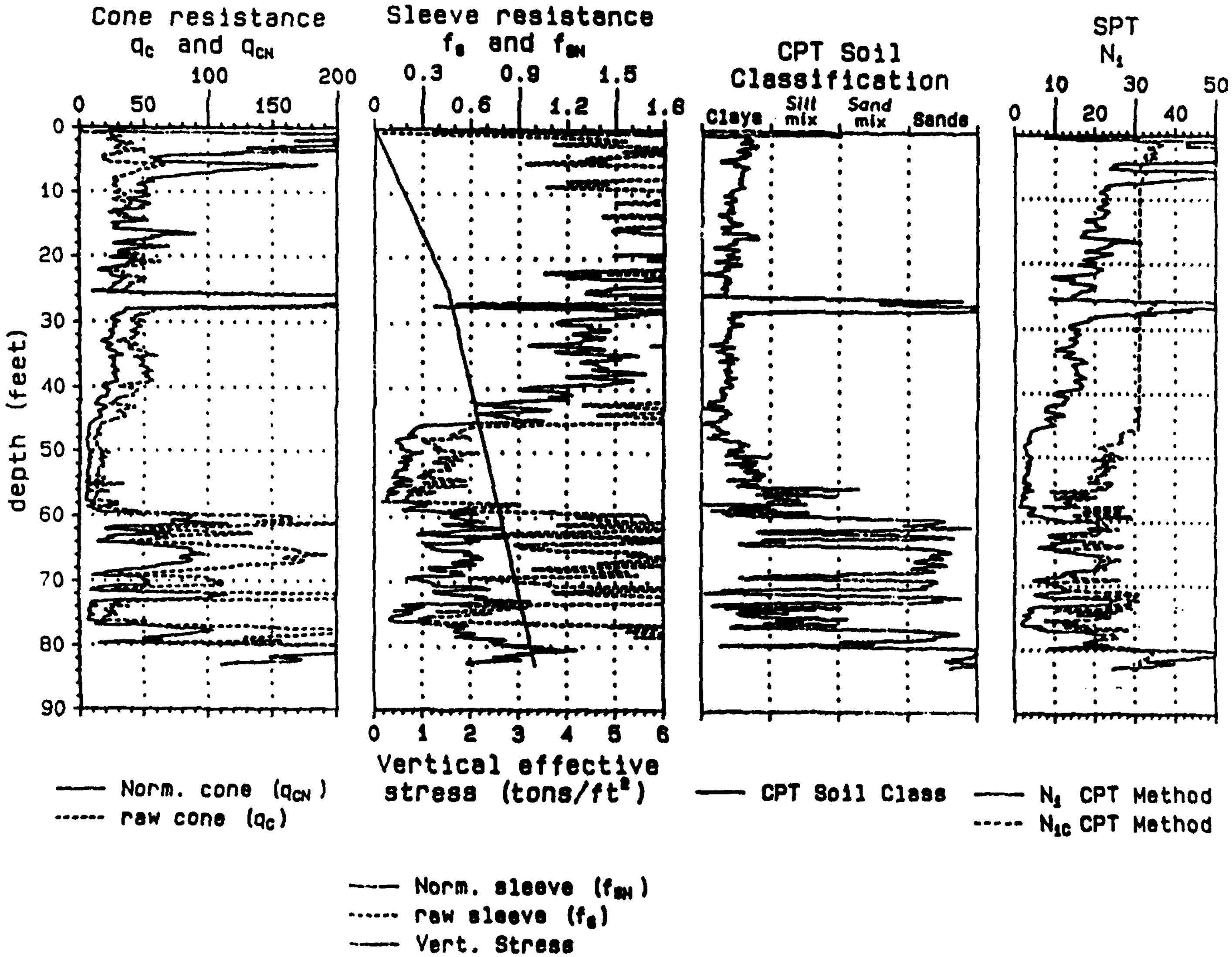
CPT Analysis

CPT-8
BARKLEY DAM



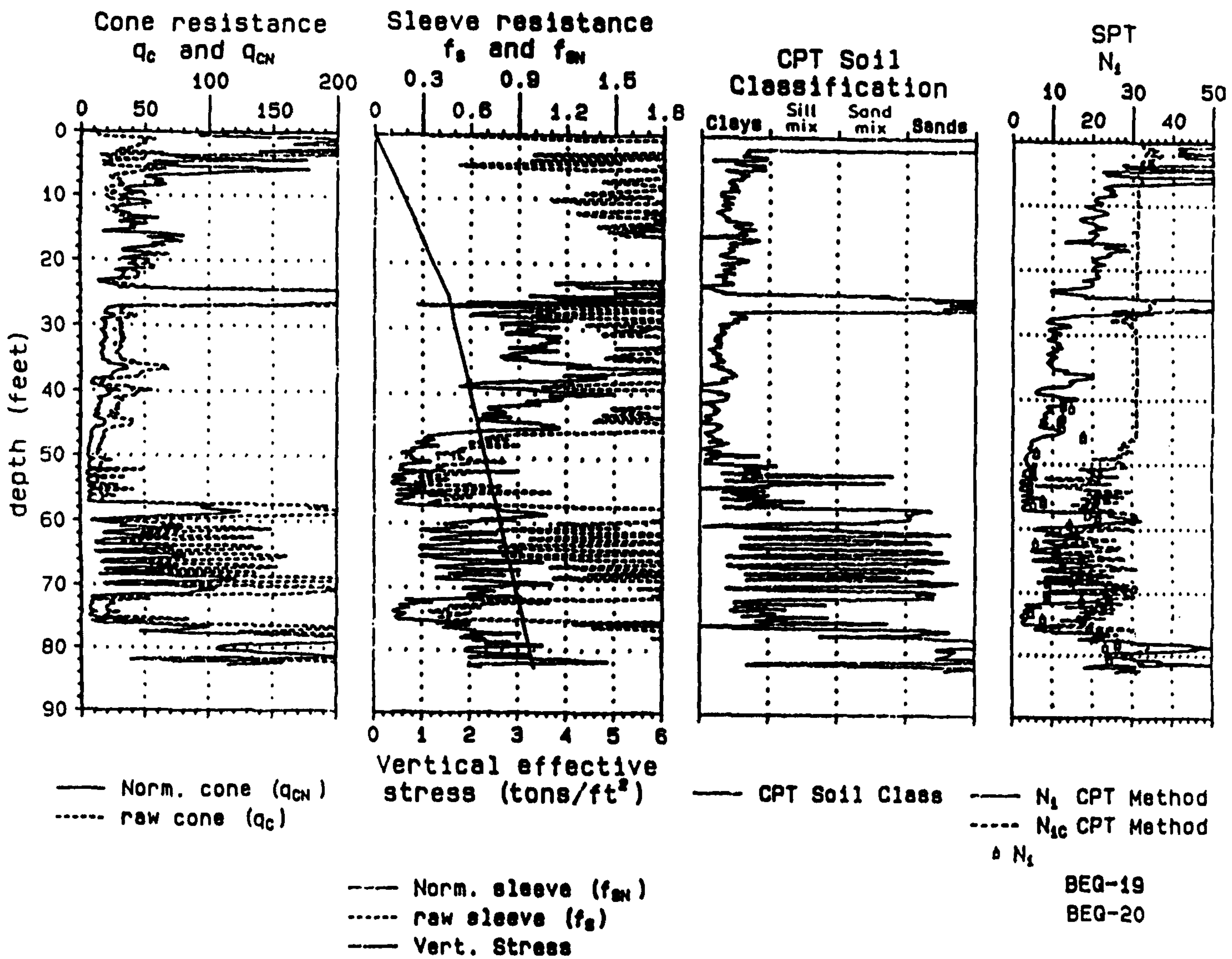
CPT Analysis

CPT-9
BARKLEY DAM



CPT Analysis

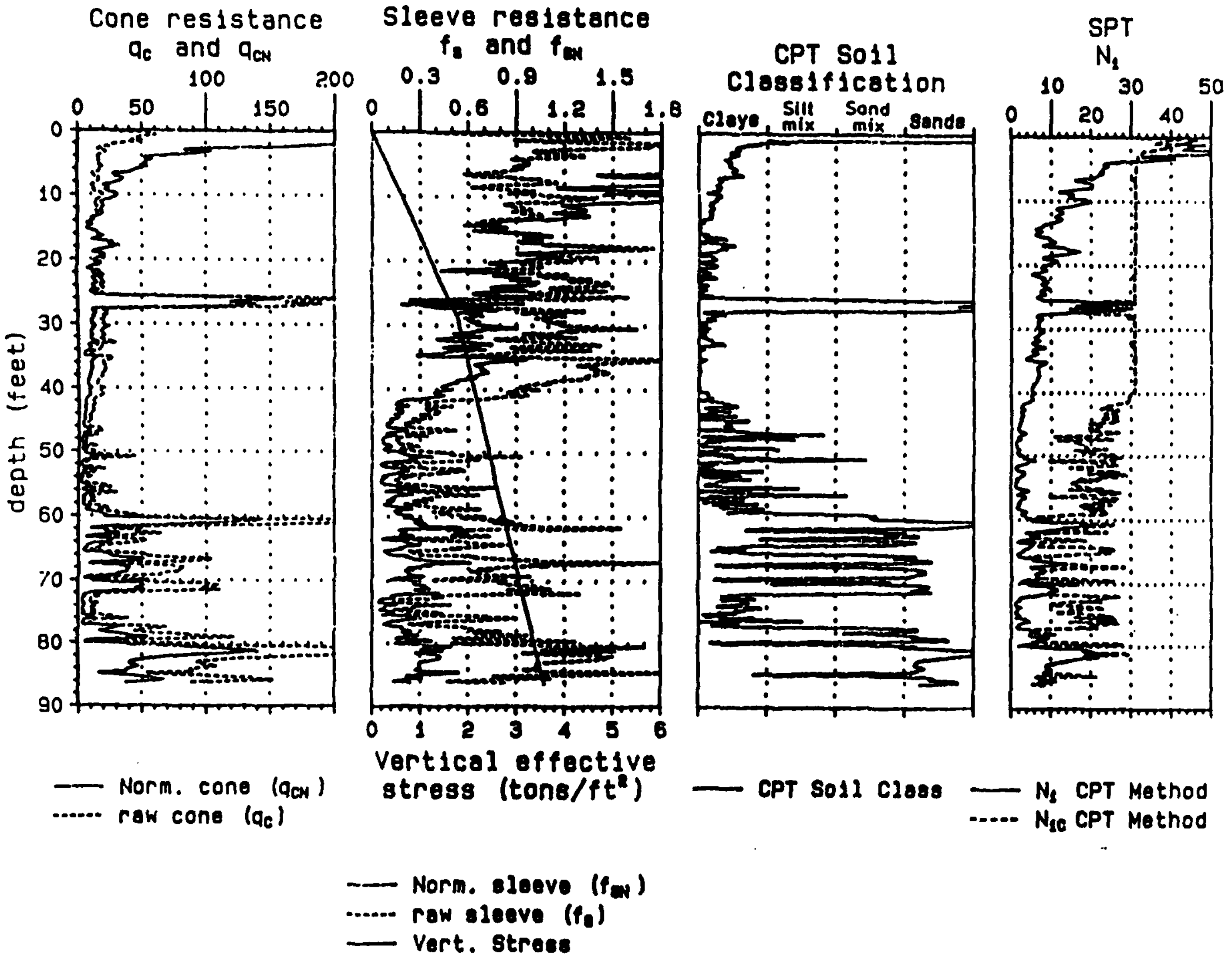
CPT-10
BARKLEY DAM



CPT Analysis

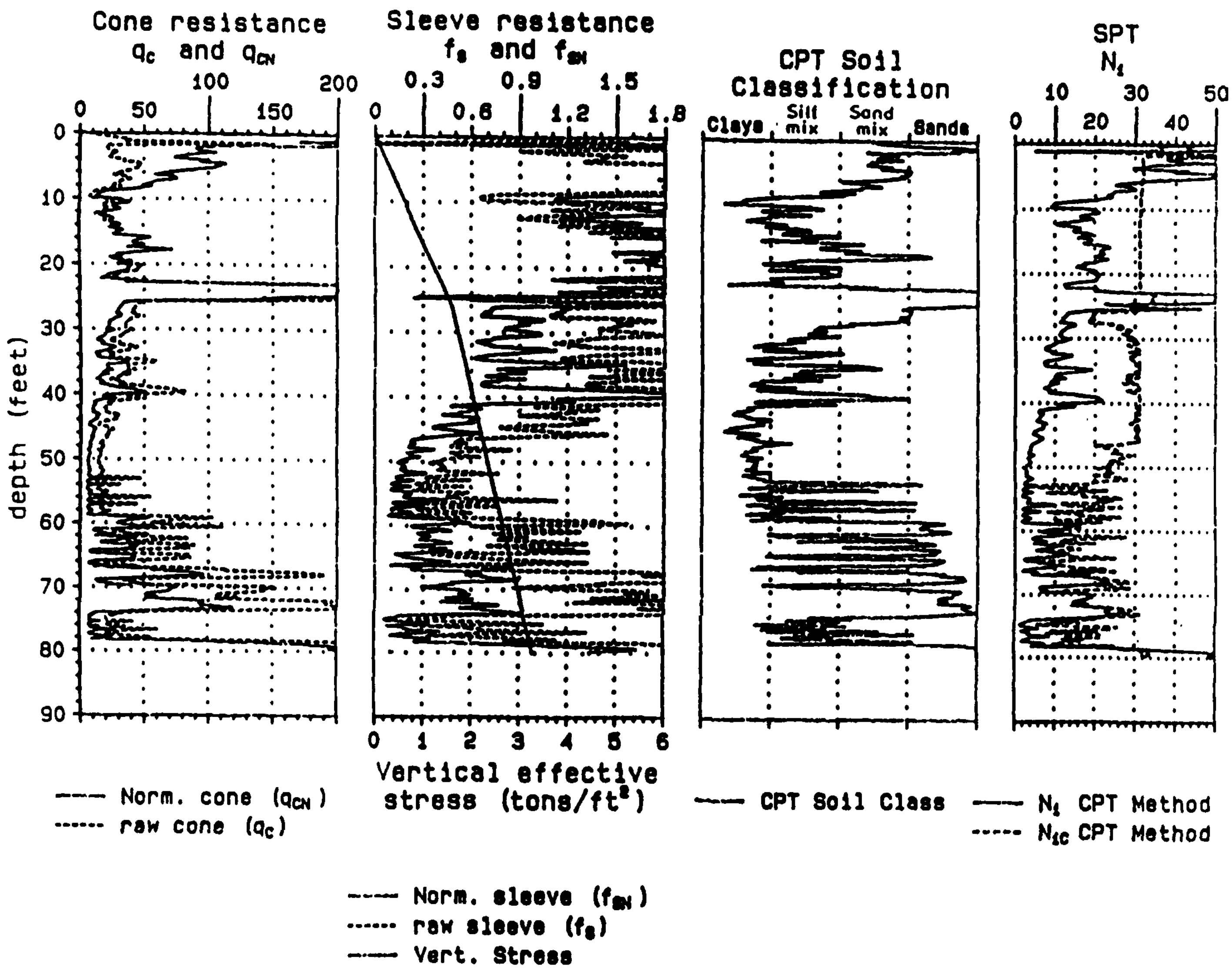
CPT-11

BARKLEY DAM



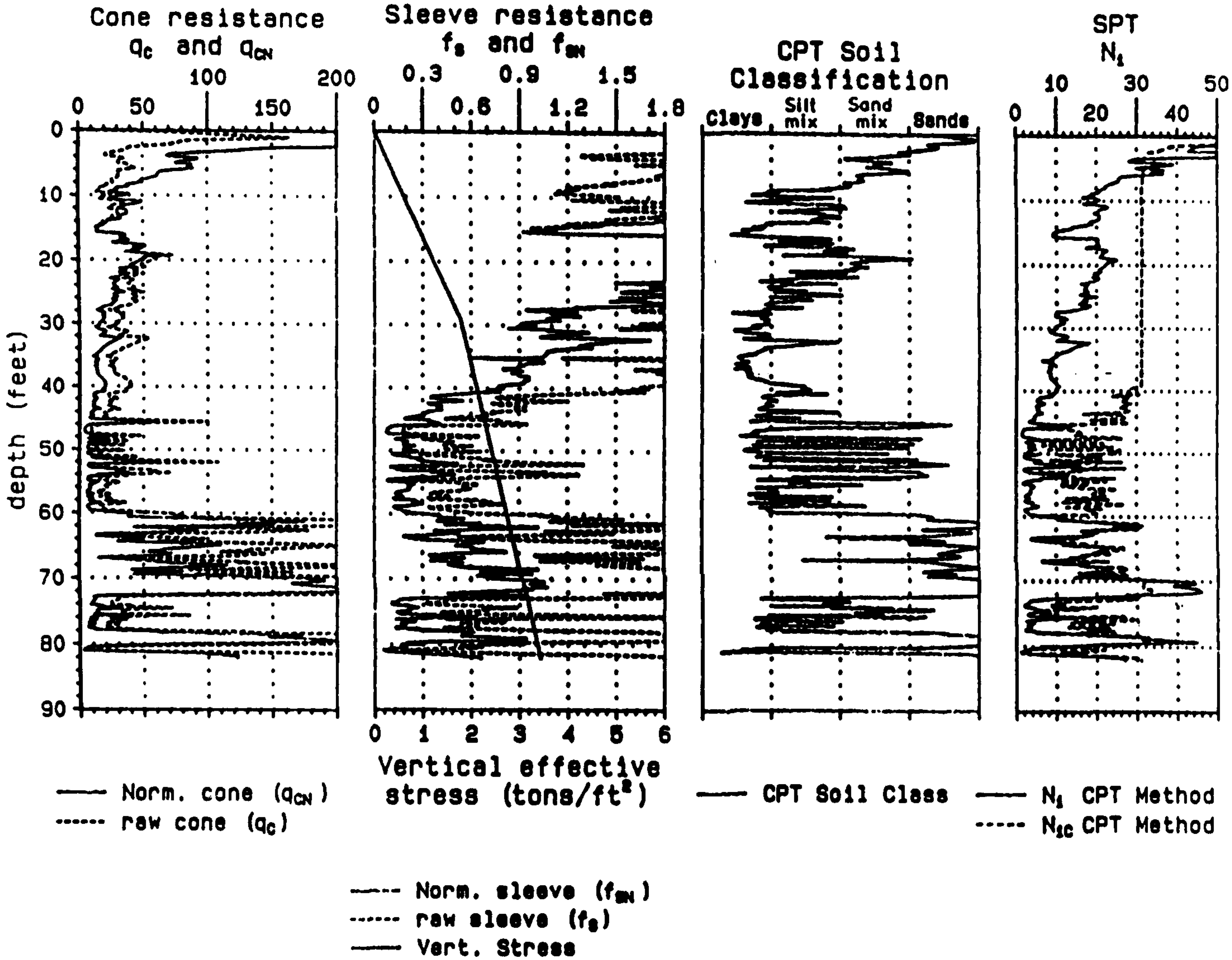
CPT Analysis

CPT-12
BARKLEY DAM



CPT Analysis

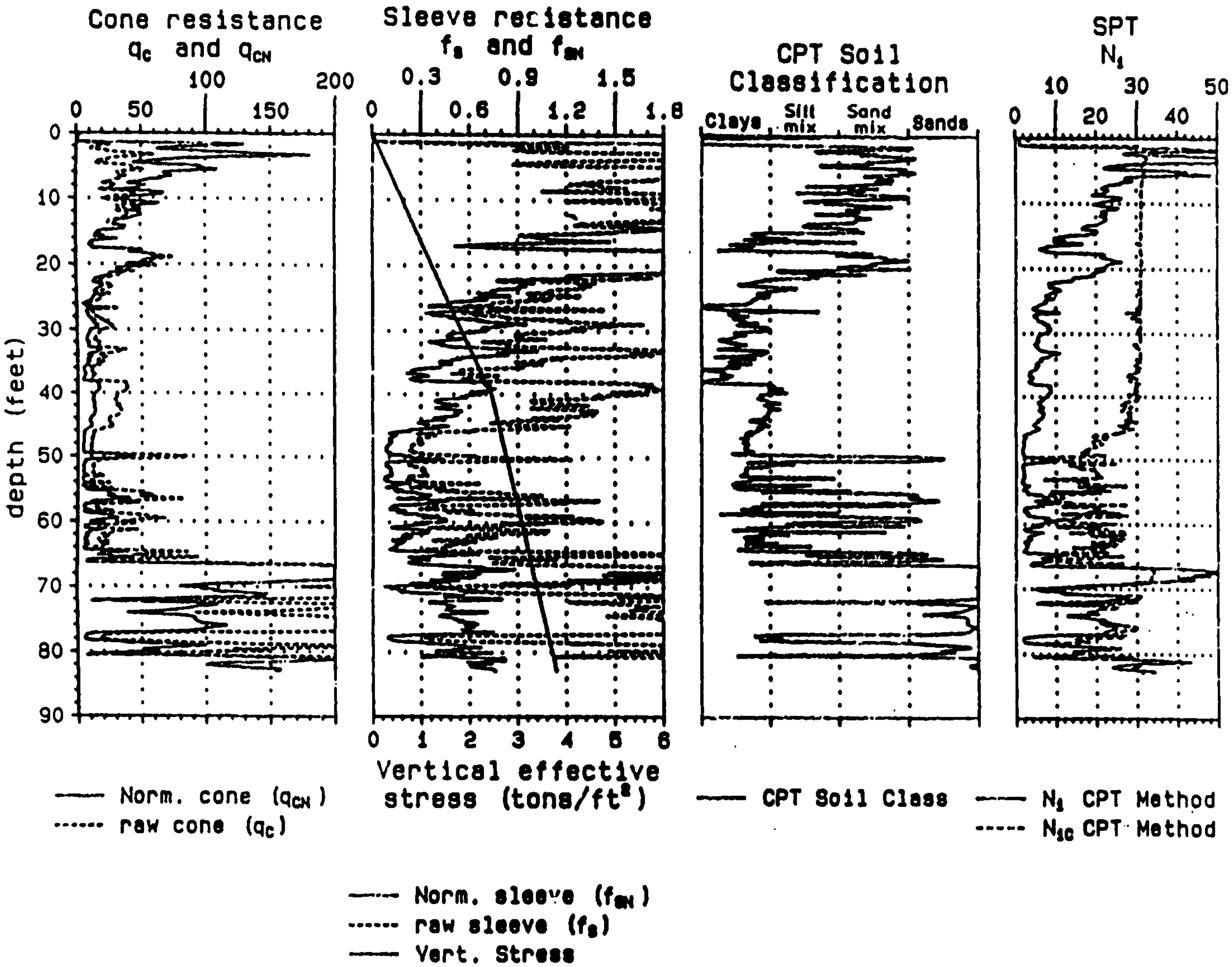
CPT-13
BARKLEY DAM



CPT Analysis

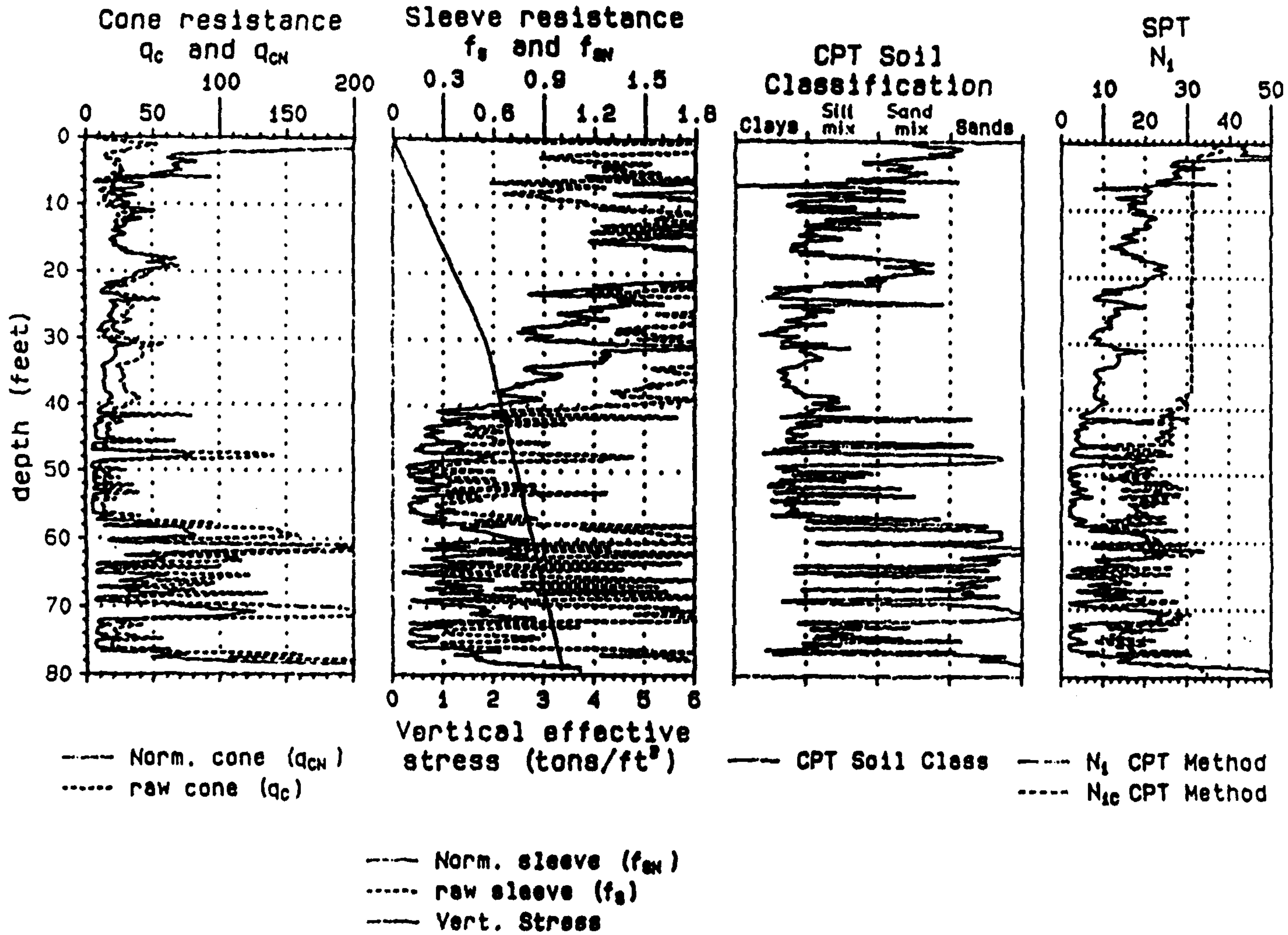
CPT-14

BARKLEY DAM



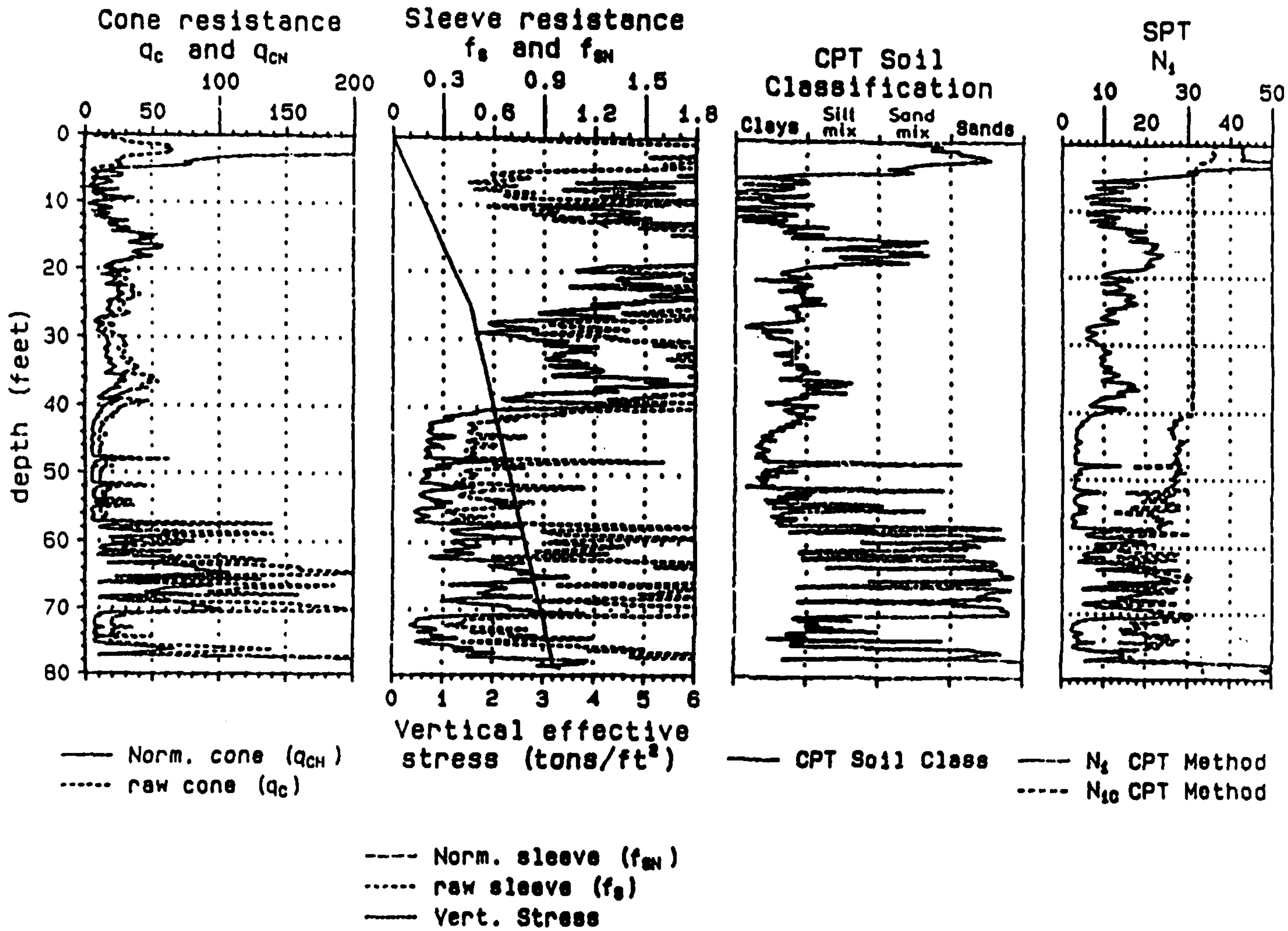
CPT Analysis

CPT-15
BARKLEY DAM



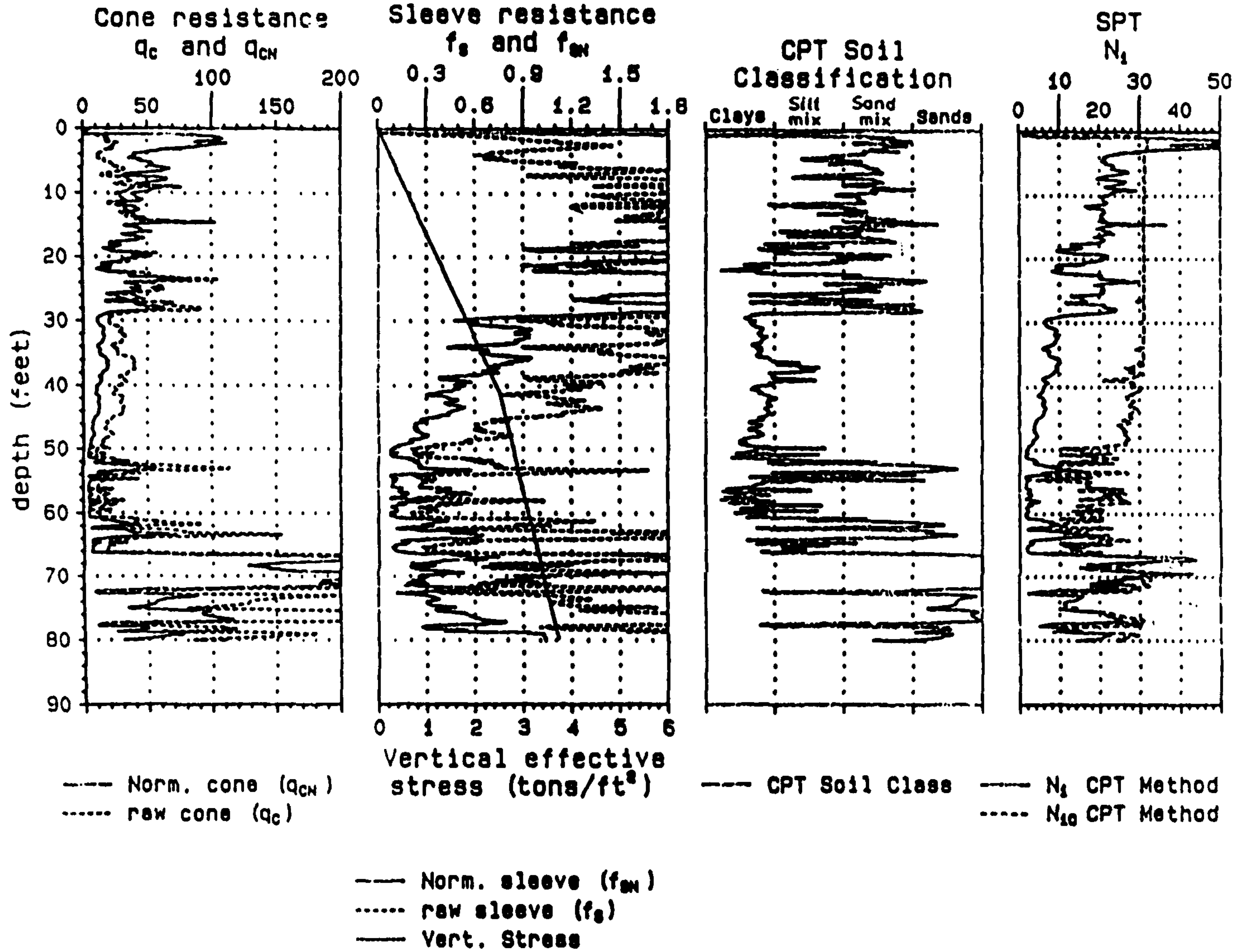
CPT Analysis

CPT-16
BARKLEY DAM



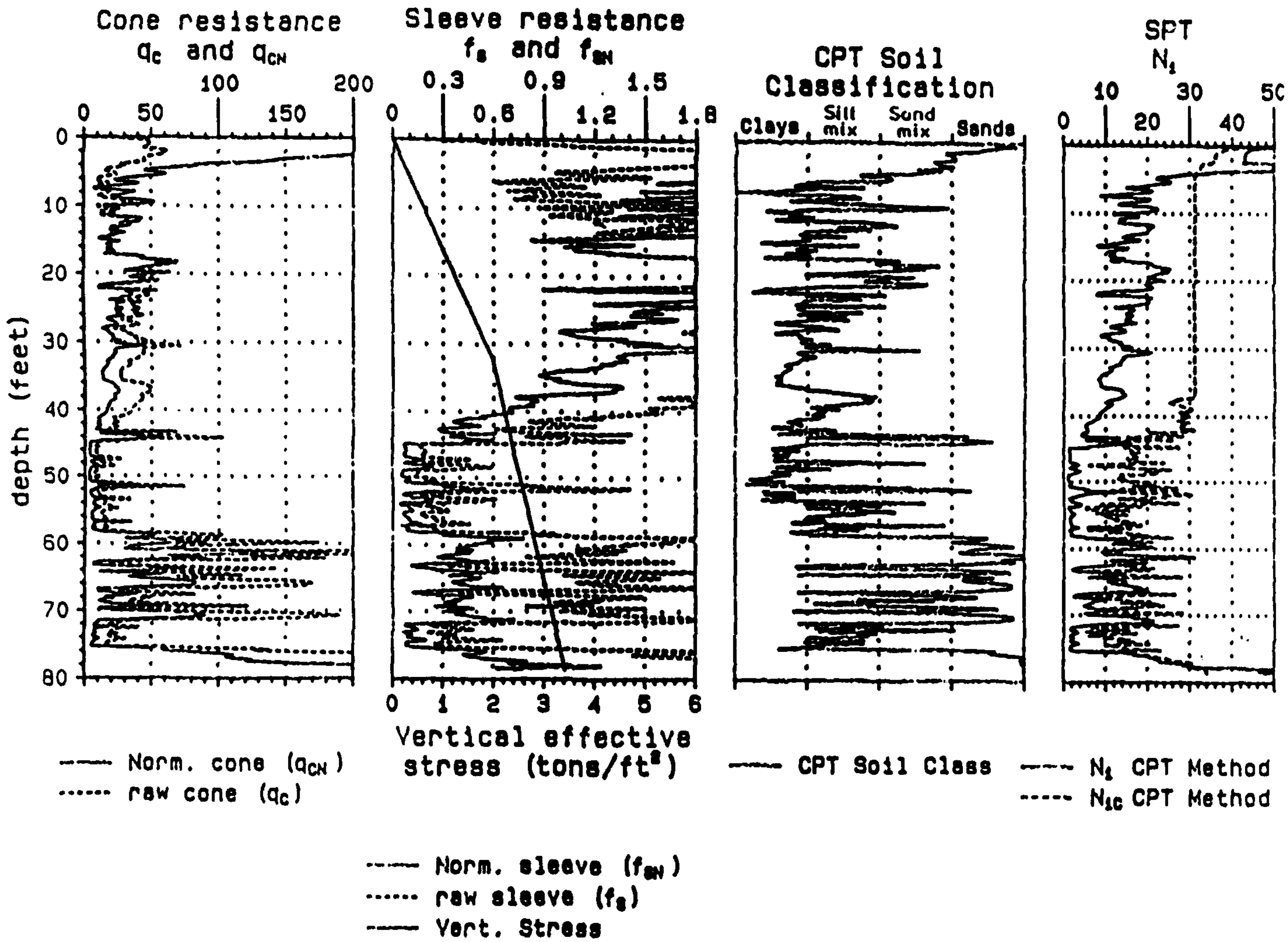
CPT Analysis

CPT-17
BARKLEY DAM



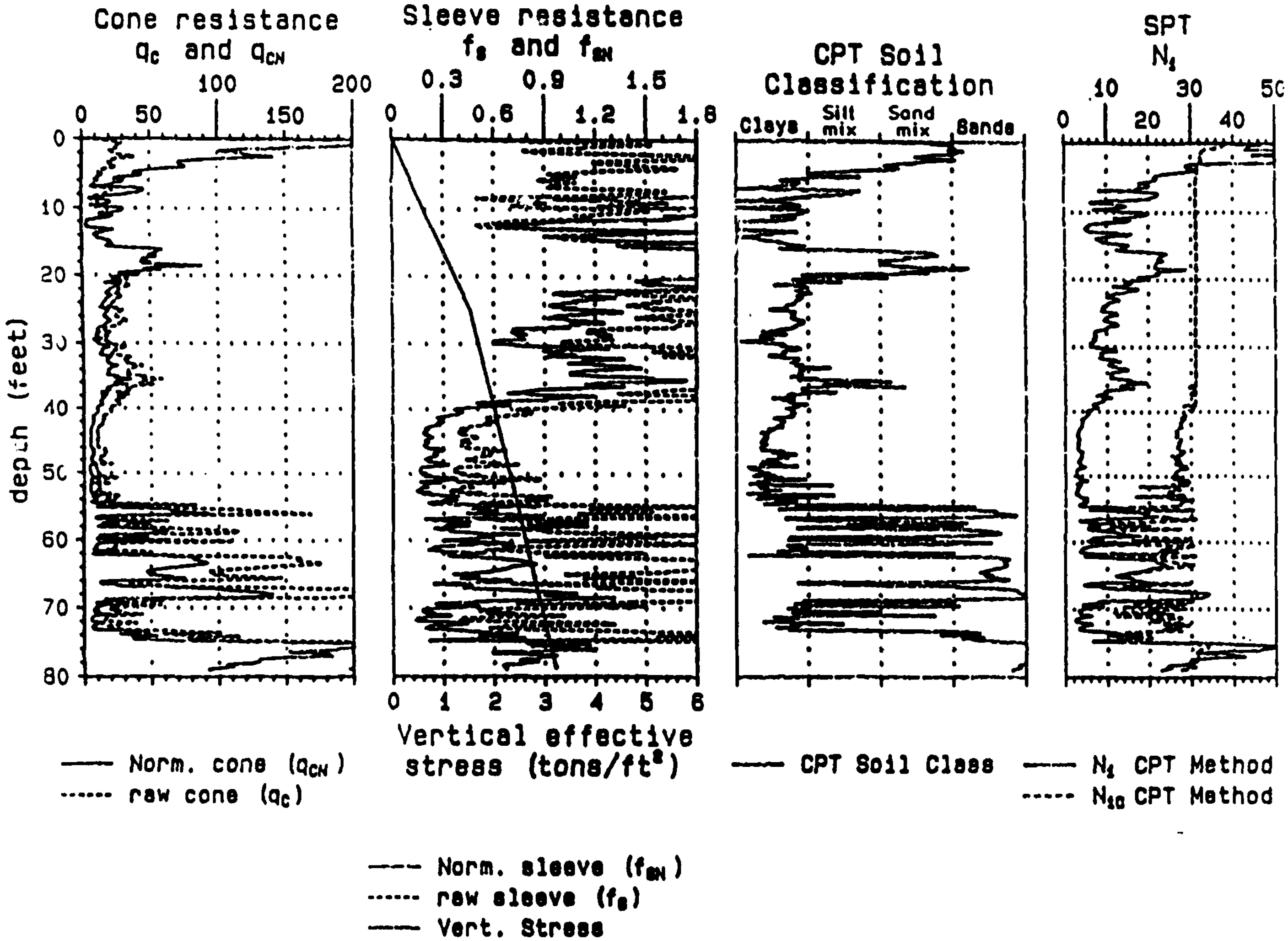
CPT Analysis

CPT-18
BARKLEY DAM



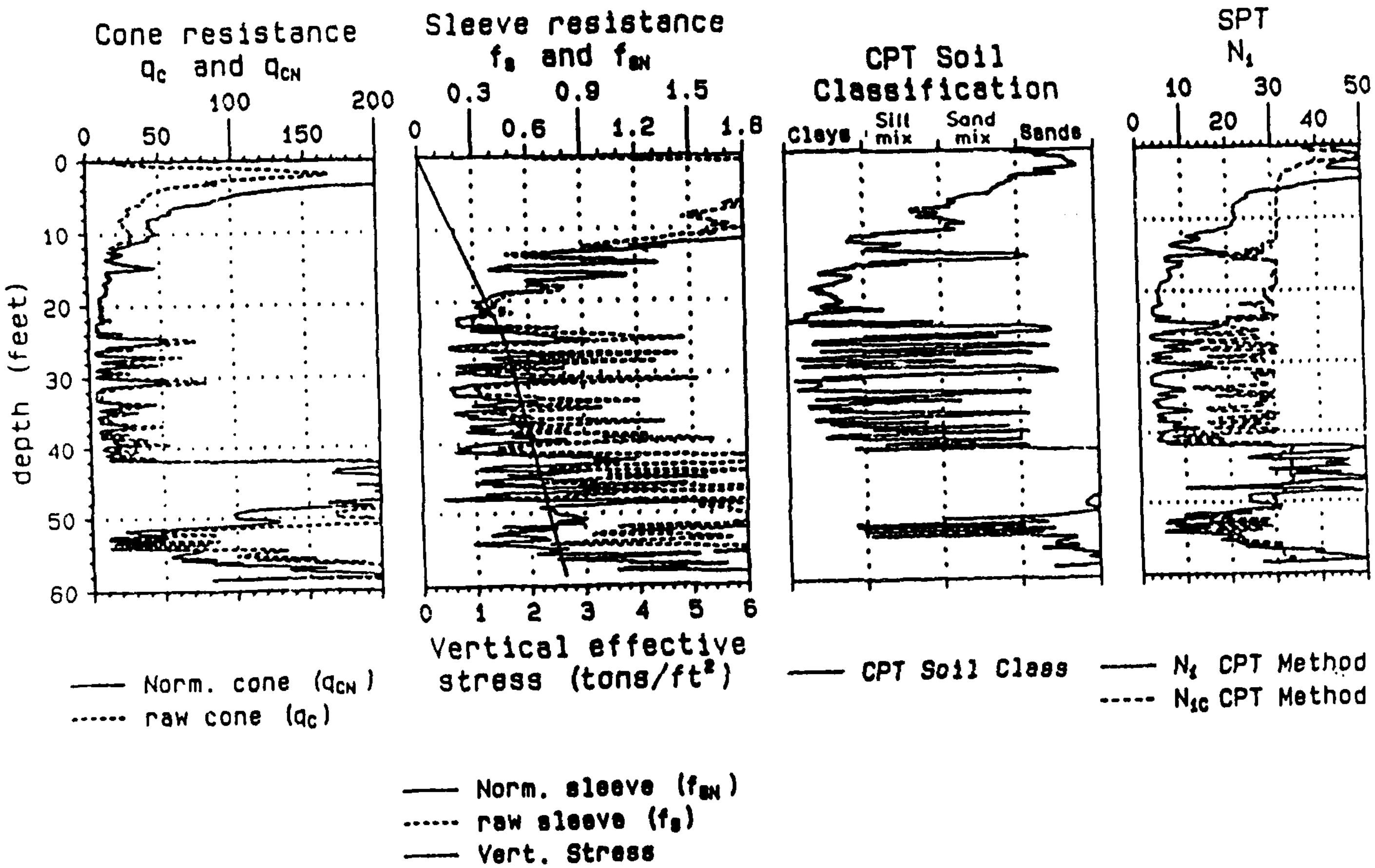
CPT Analysis

CPT-19
BARKLEY DAM



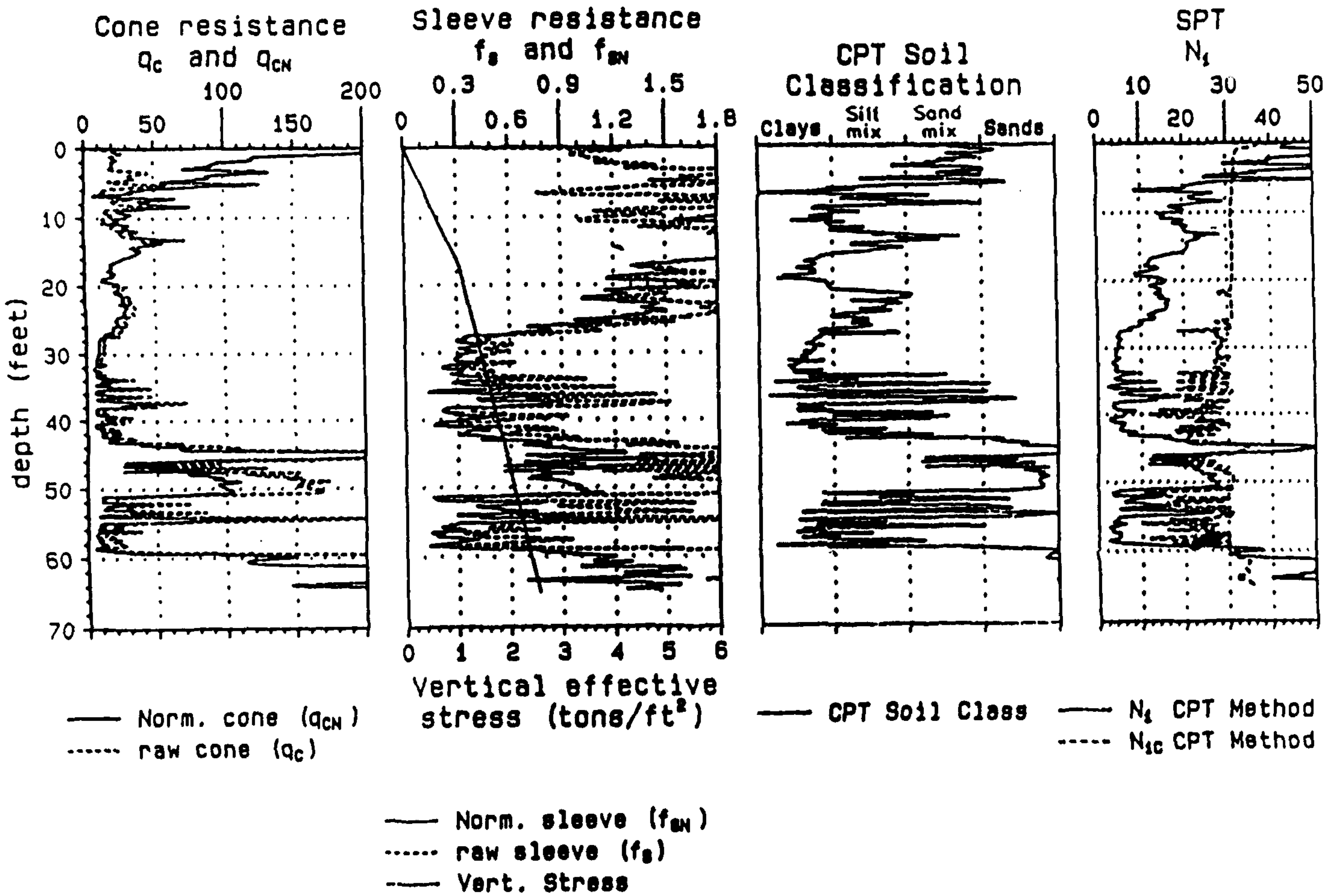
CPT Analysis

CPT-20
BARKLEY DAM



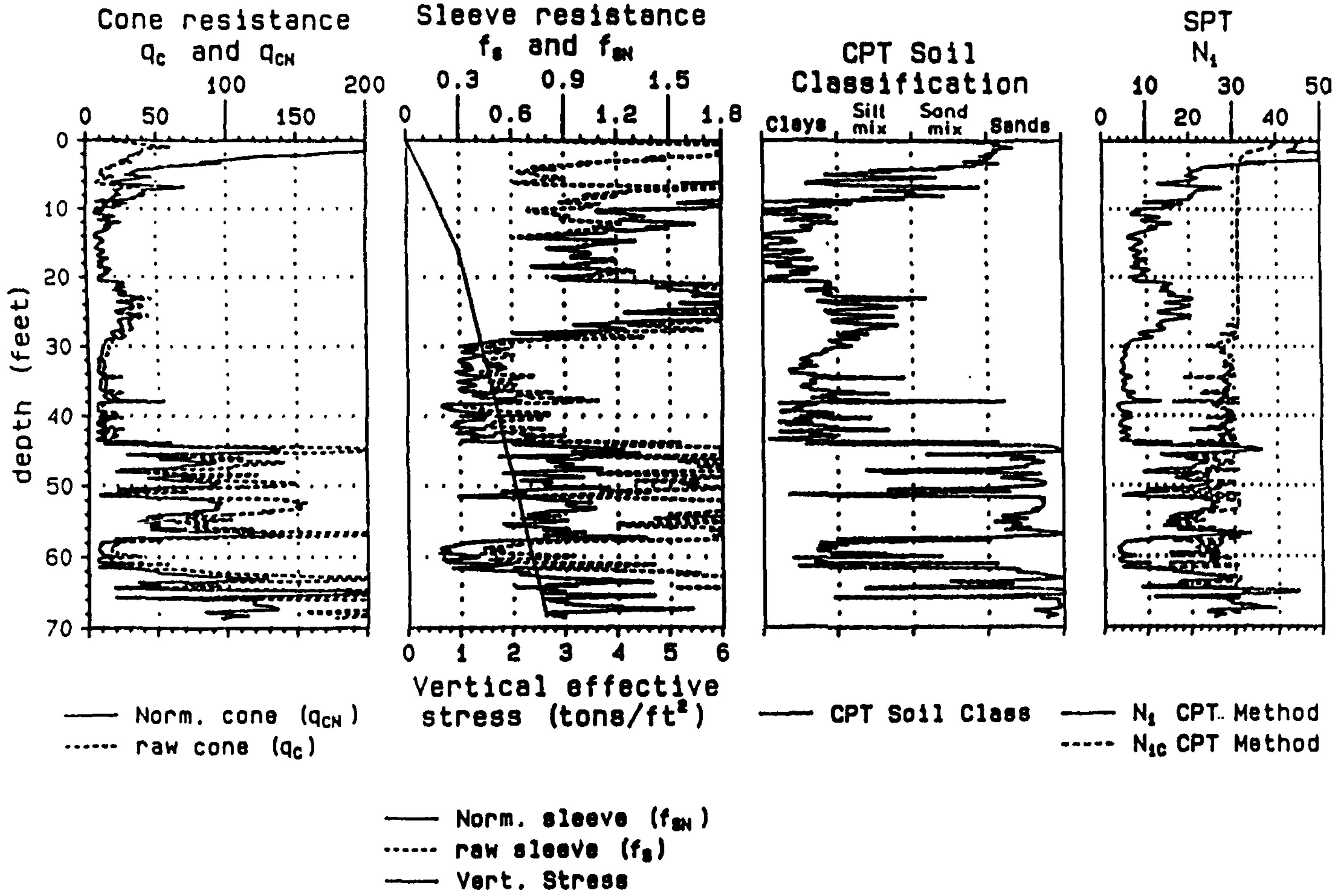
CPT Analysis

CPT-21
BARKLEY DAM



CPT Analysis

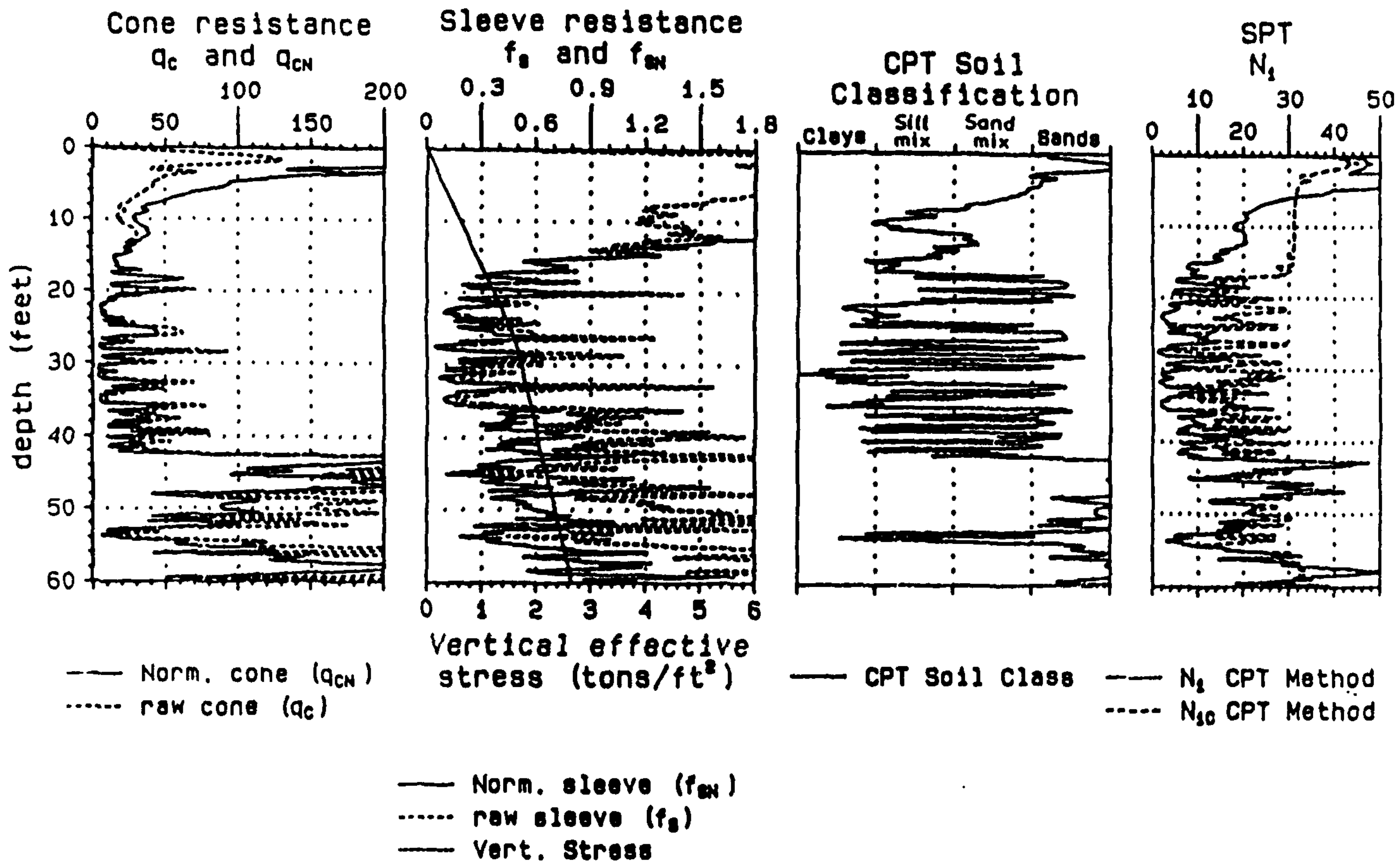
CPT-22
BARKLEY DAM



B24

CPT Analysis

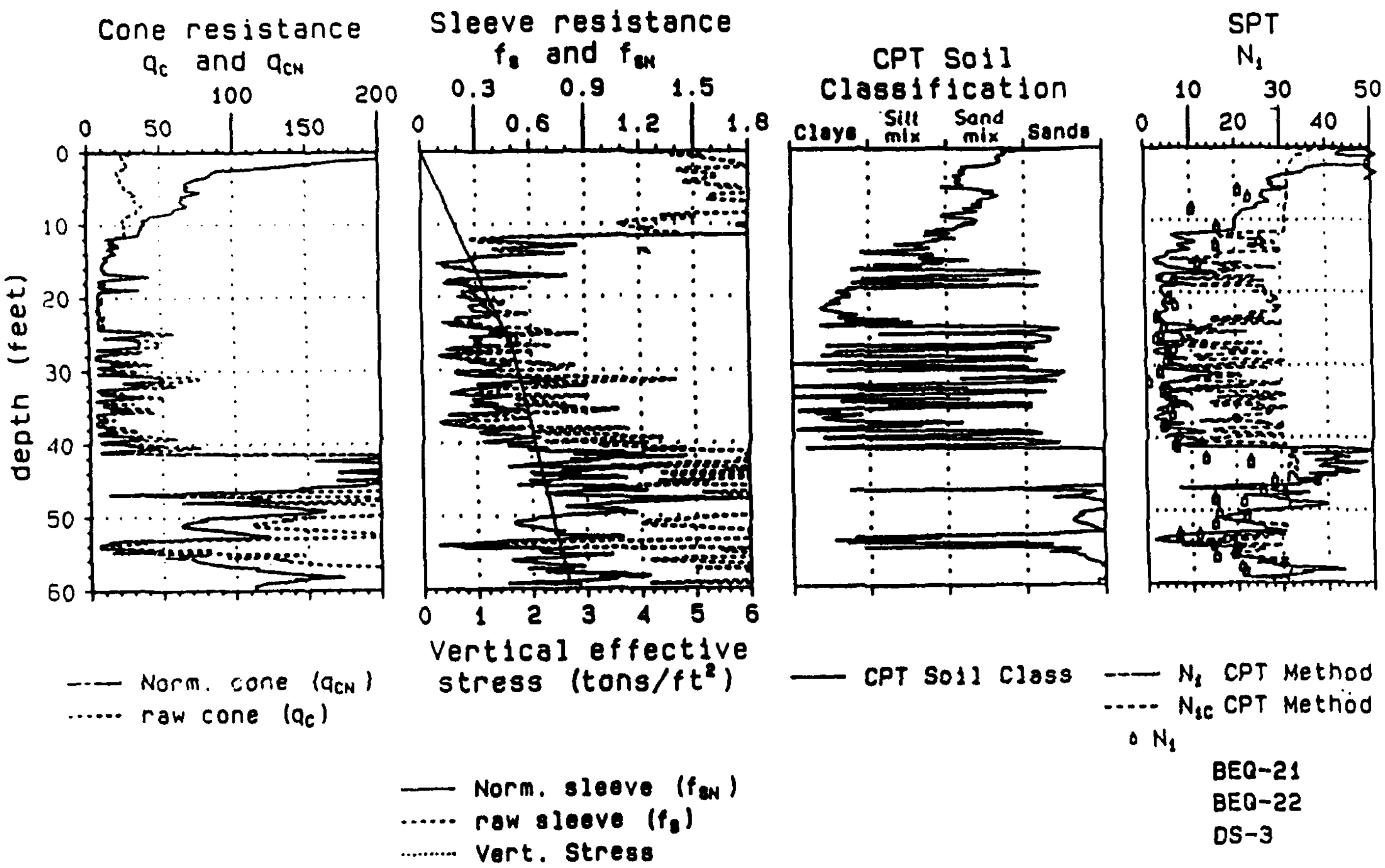
CPT-23
BARKLEY DAM



CPT Analysis

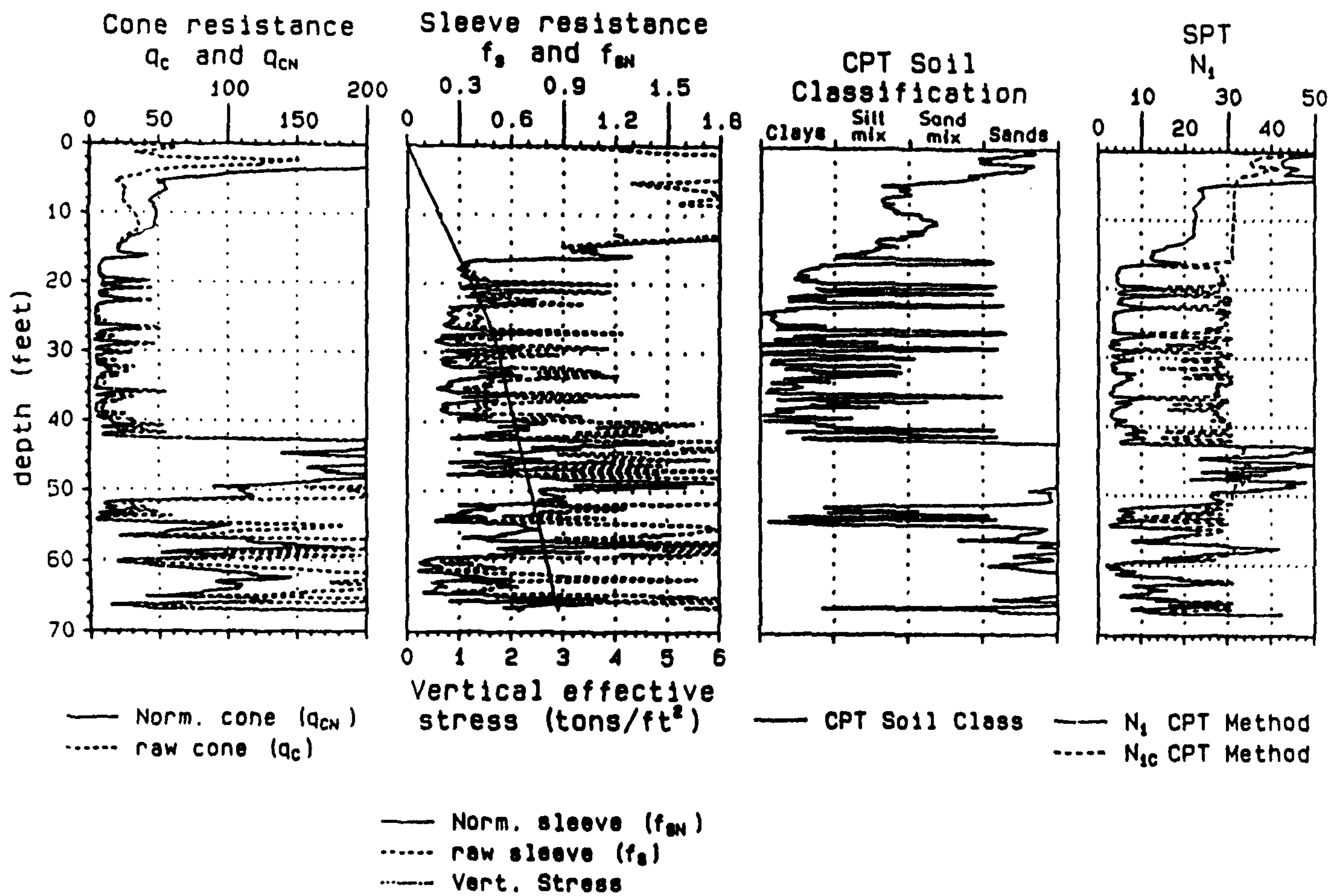
CPT-24

BARKLEY DAM



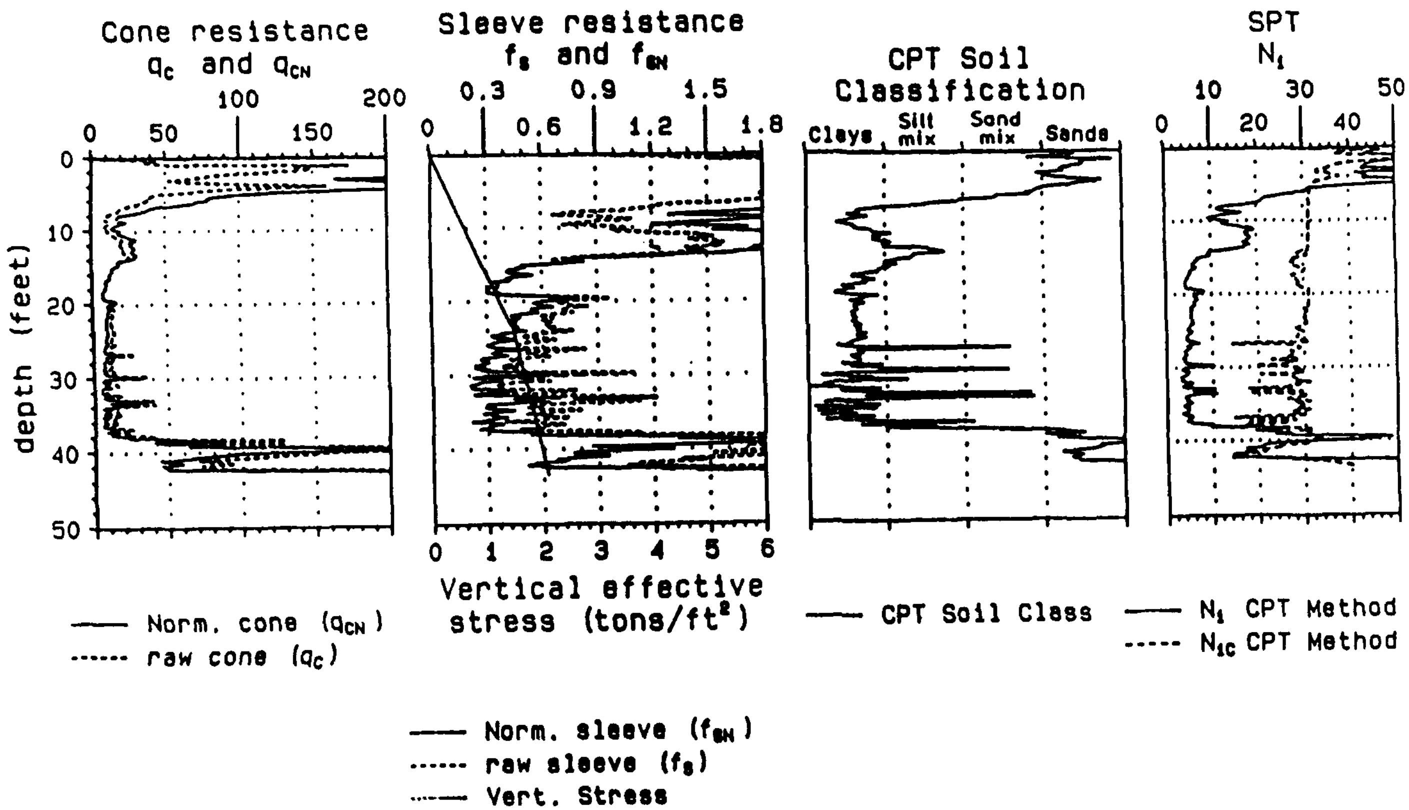
CPT Analysis

CPT-25
BARKLEY DAM



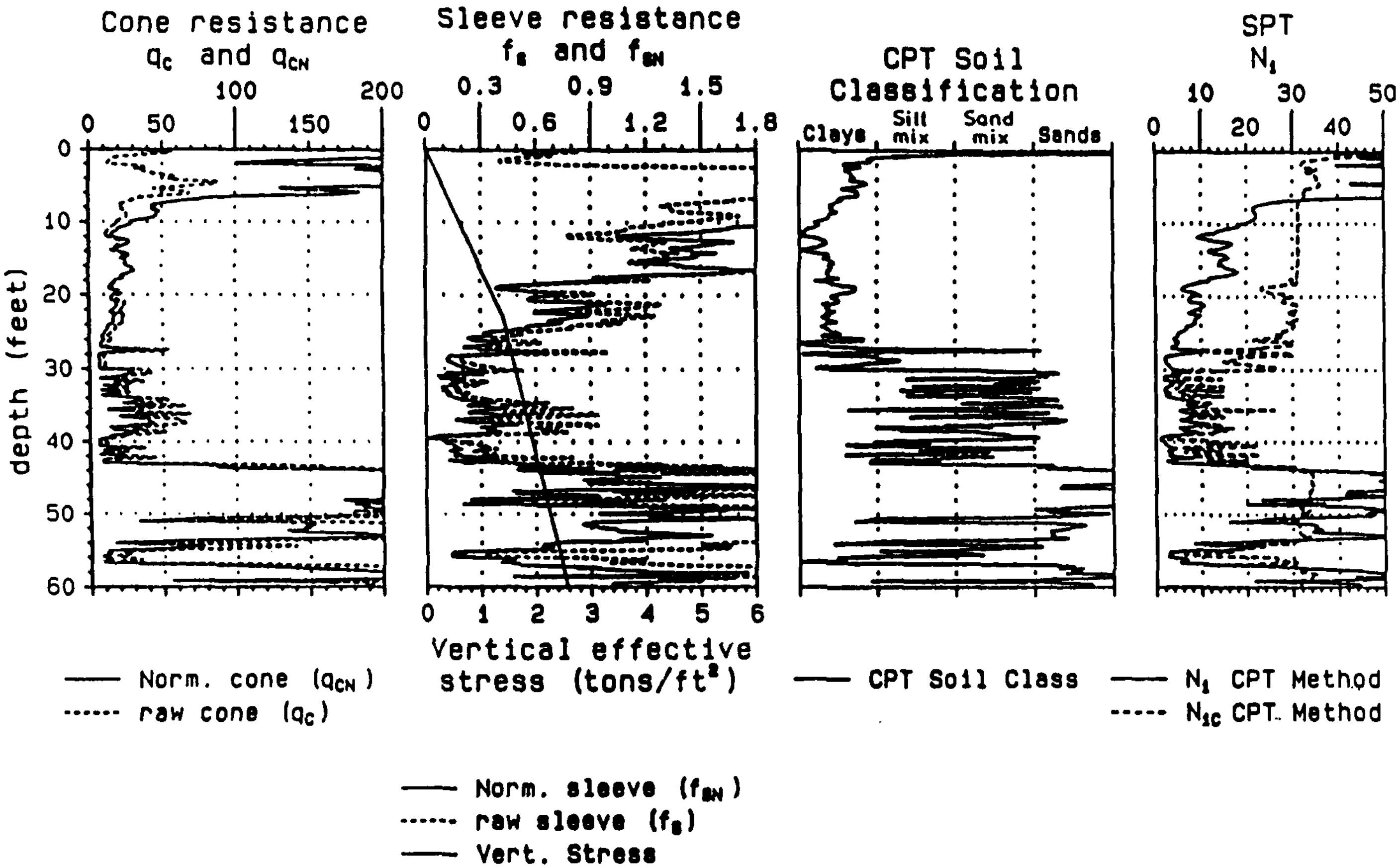
CPT Analysis

CPT-26
BARKLEY DAM



CPT Analysis

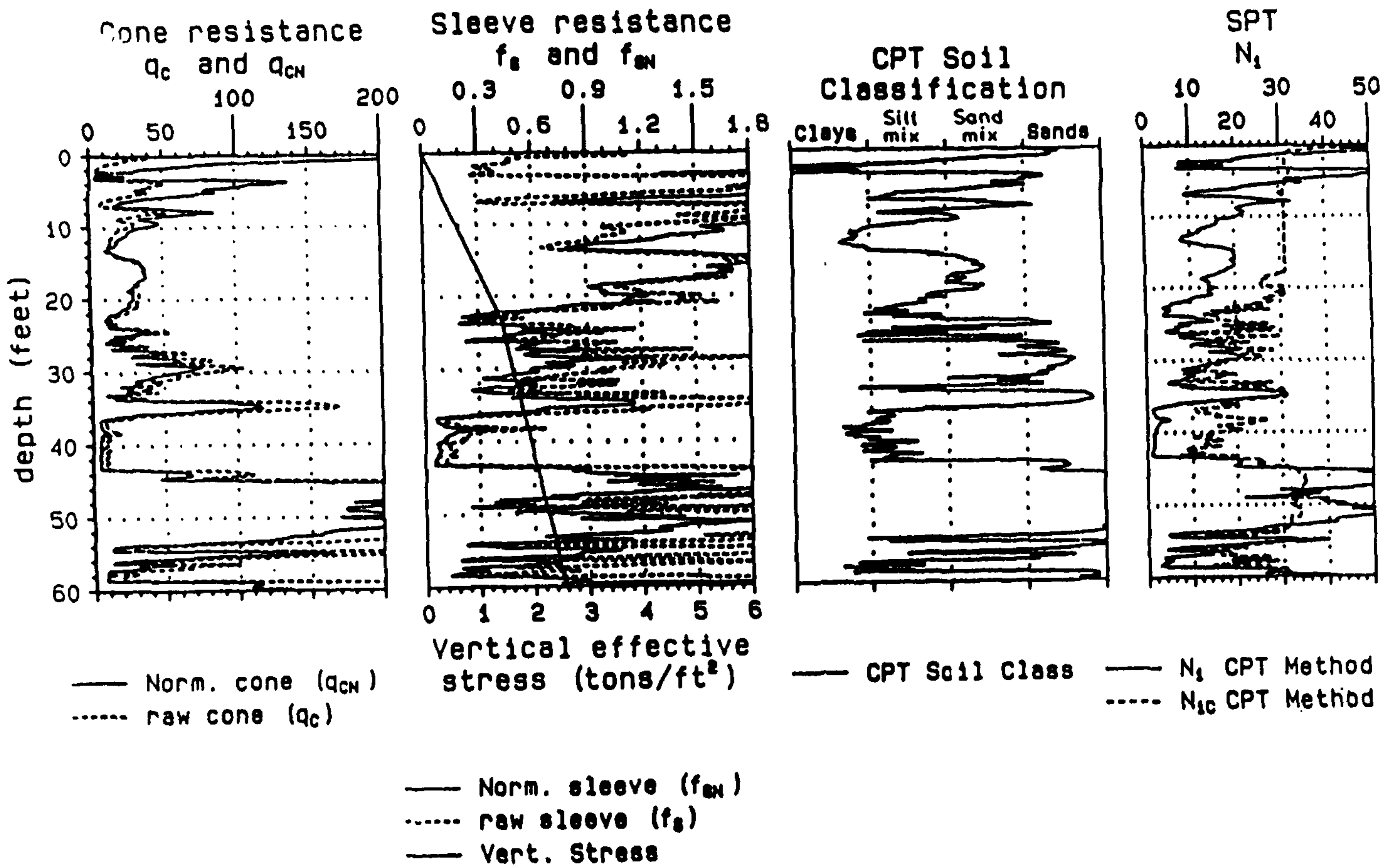
CPT-27
BARKLEY DAM



CPT Analysis

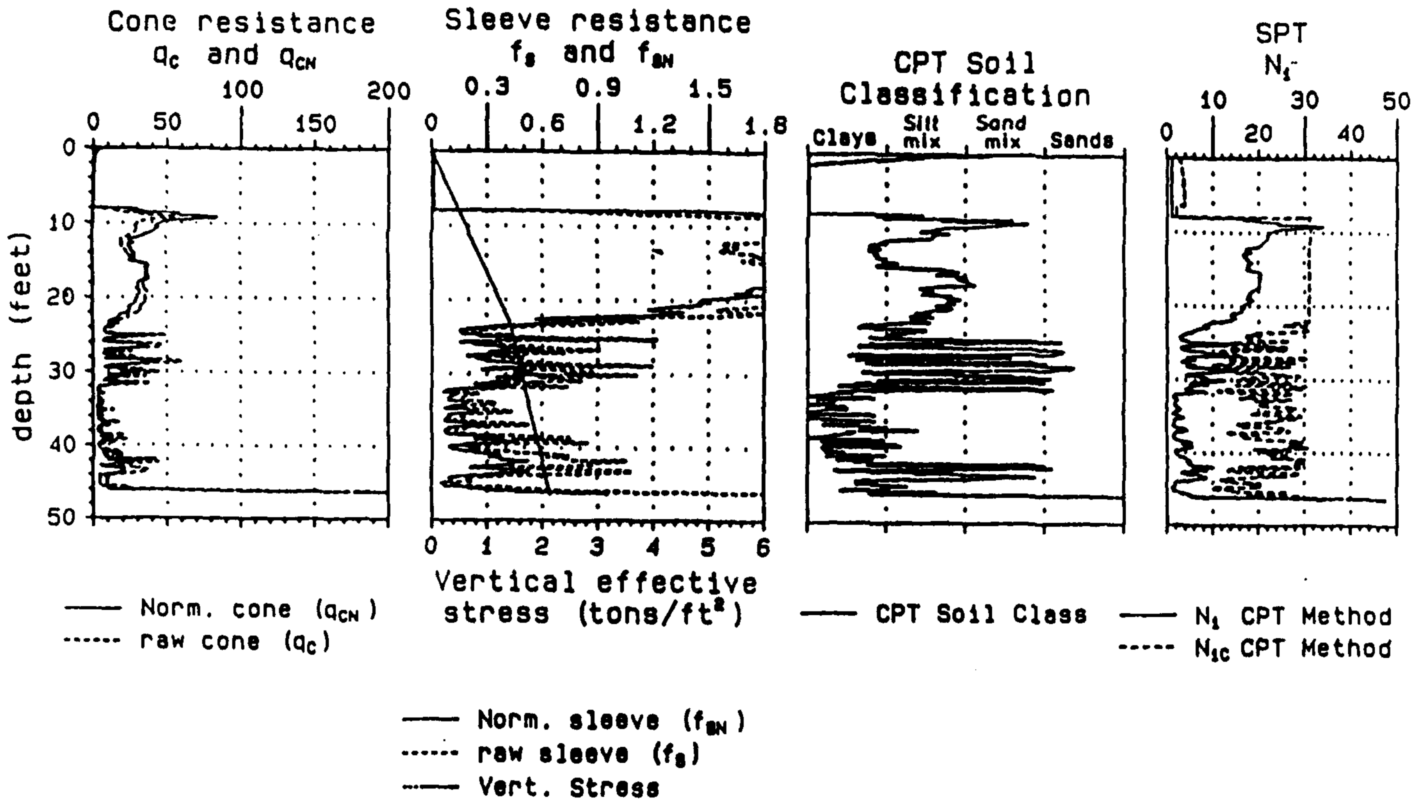
CPT-28

BARKLEY DAM



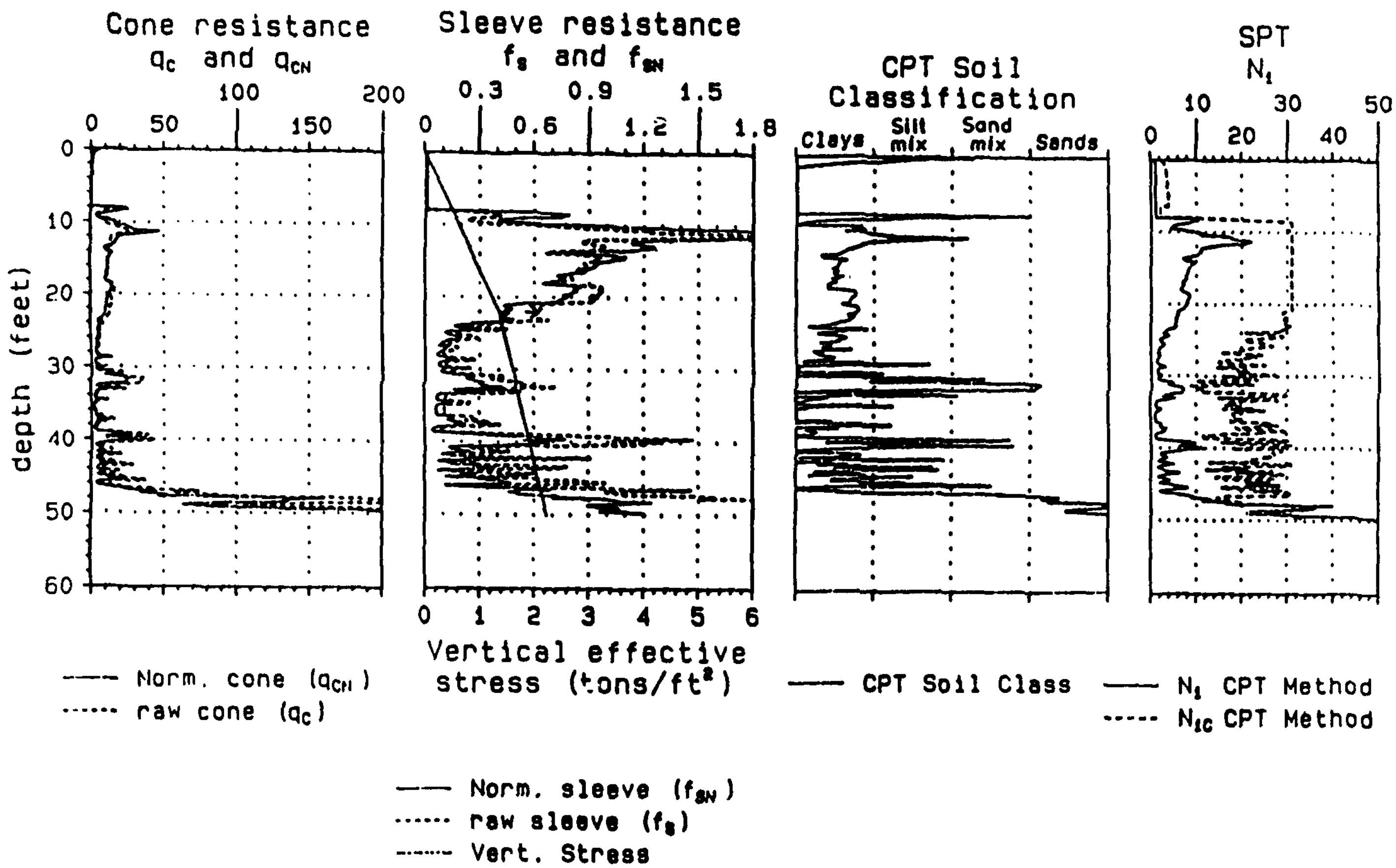
CPT Analysis

CPT-29
BARKLEY DAM



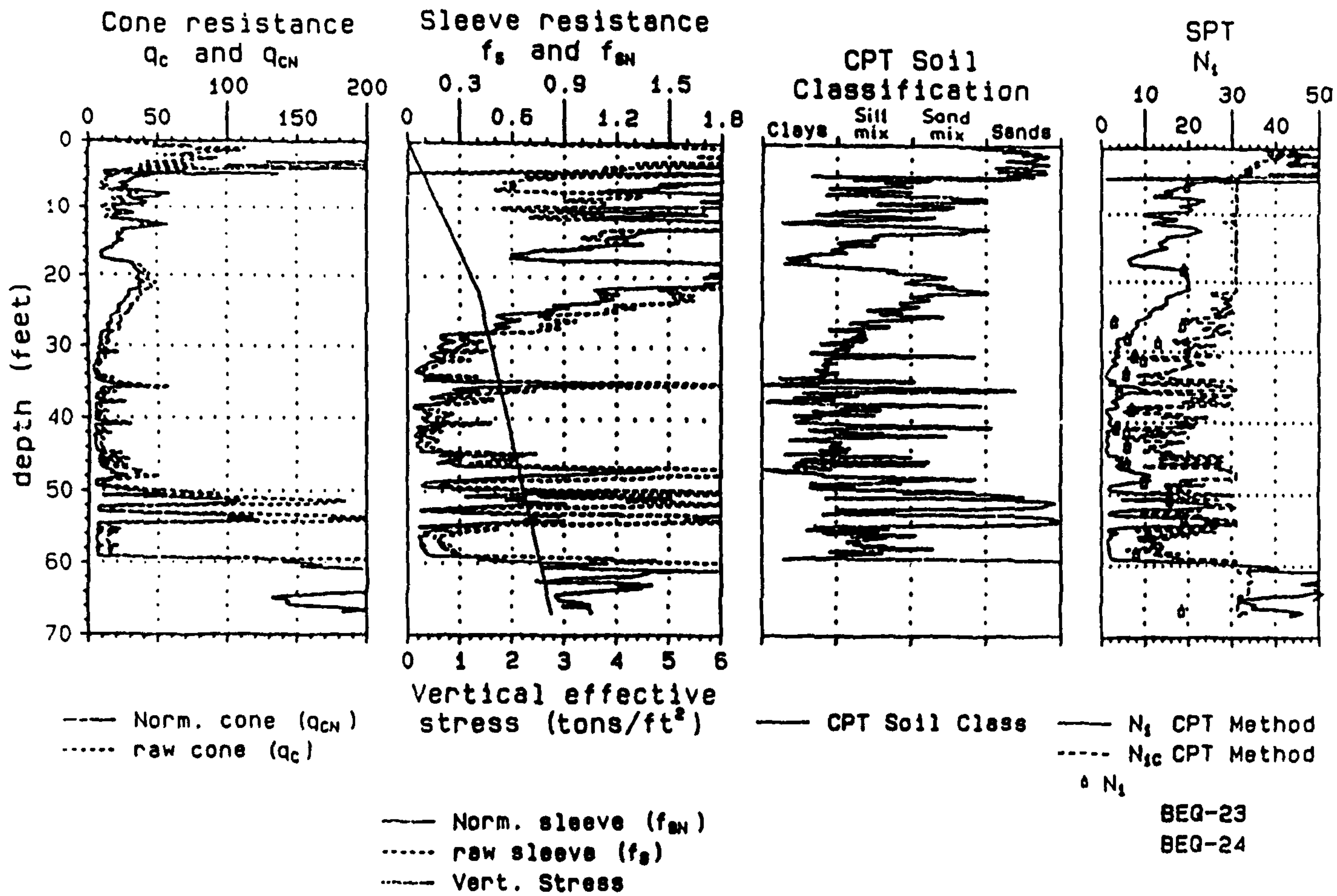
CPT Analysis

CPT-30
BARKLEY DAM



CPT Analysis

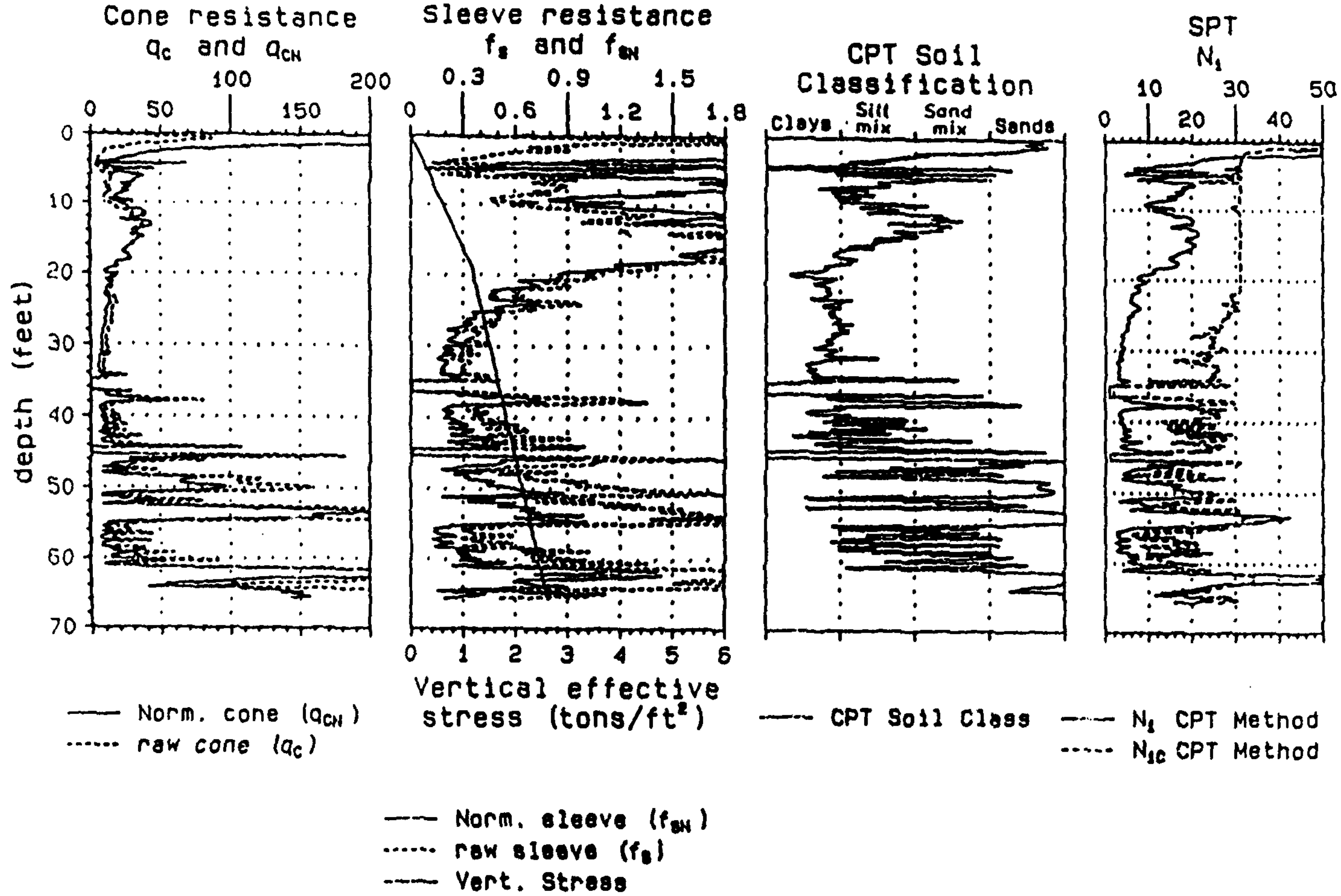
CPT-32
BARKLEY DAM



CPT Analysis

CPT-33

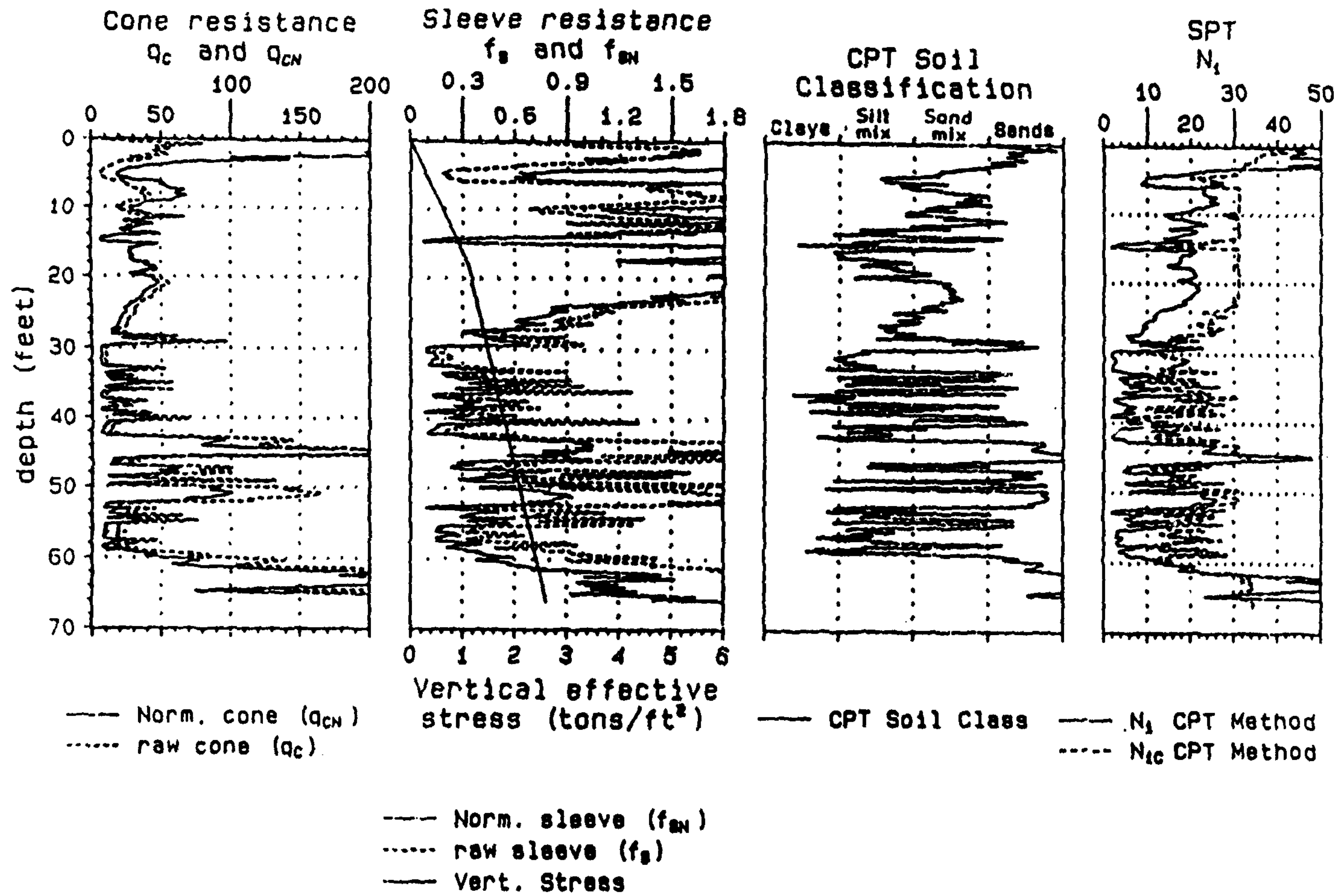
BARKLEY DAM



CPT Analysis

CPT-34
BARKLEY DAM

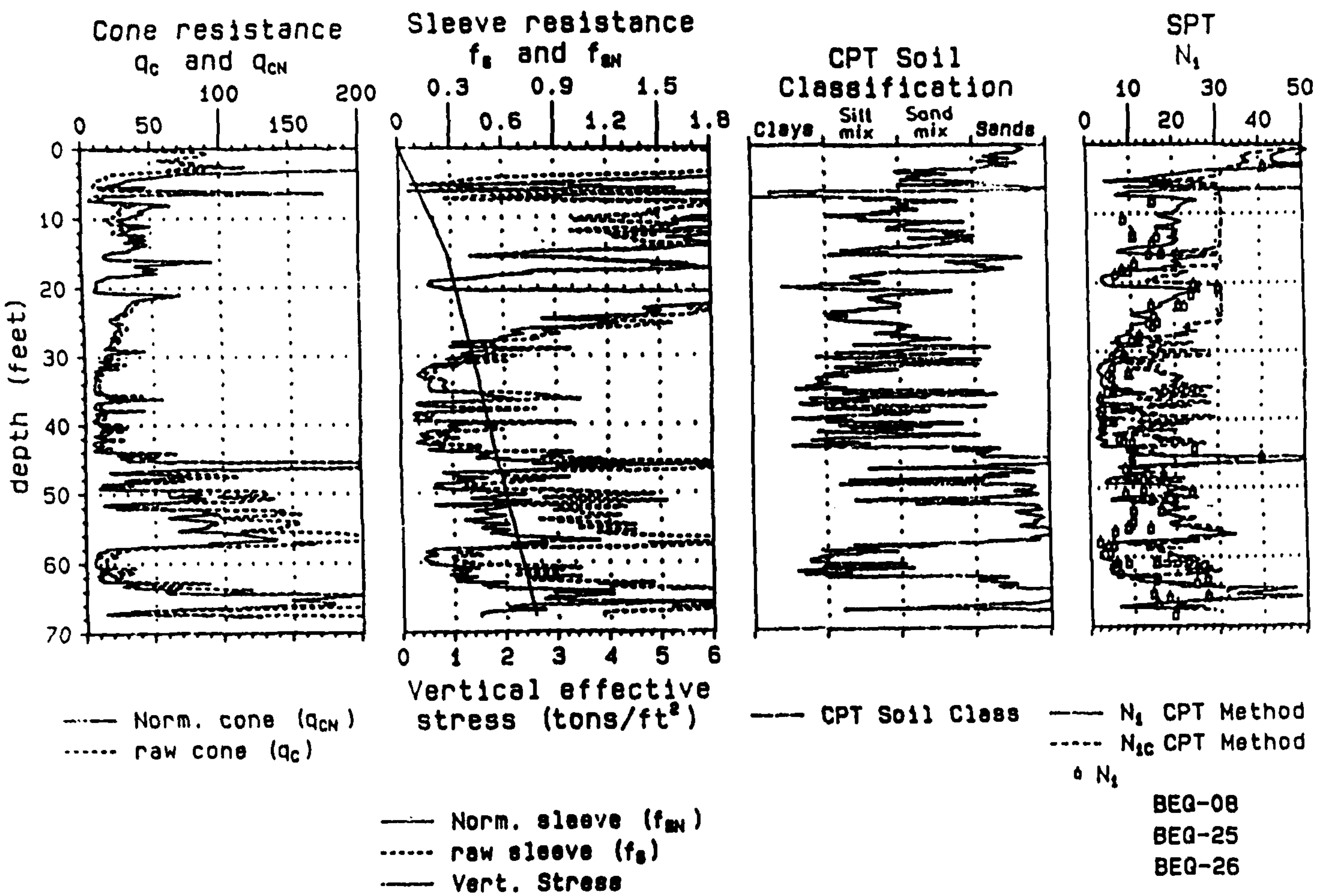
B3



CPT Analysis

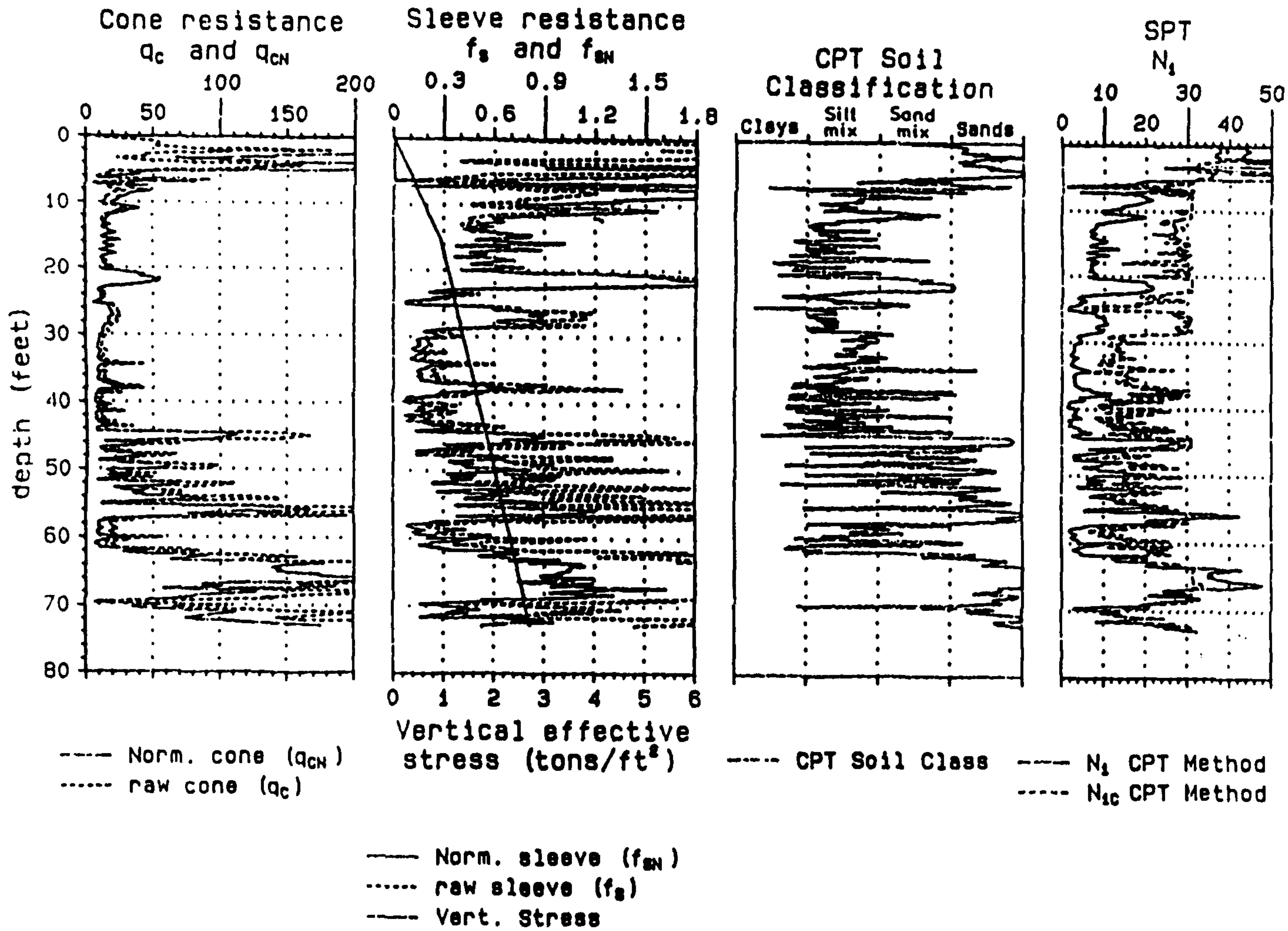
CPT-35

BARKLEY DAM



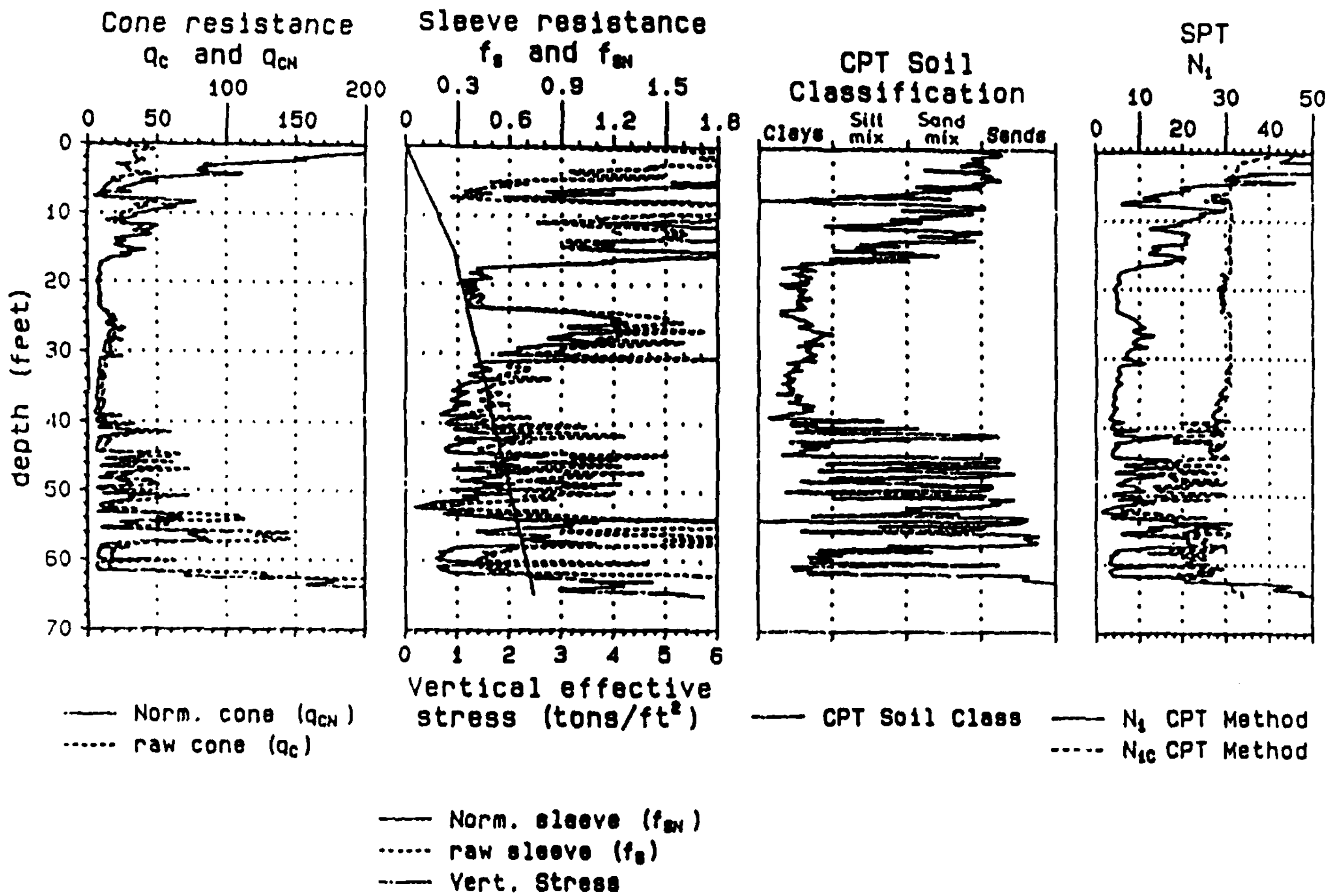
CPT Analysis

CPT-36
BARKLEY DAM



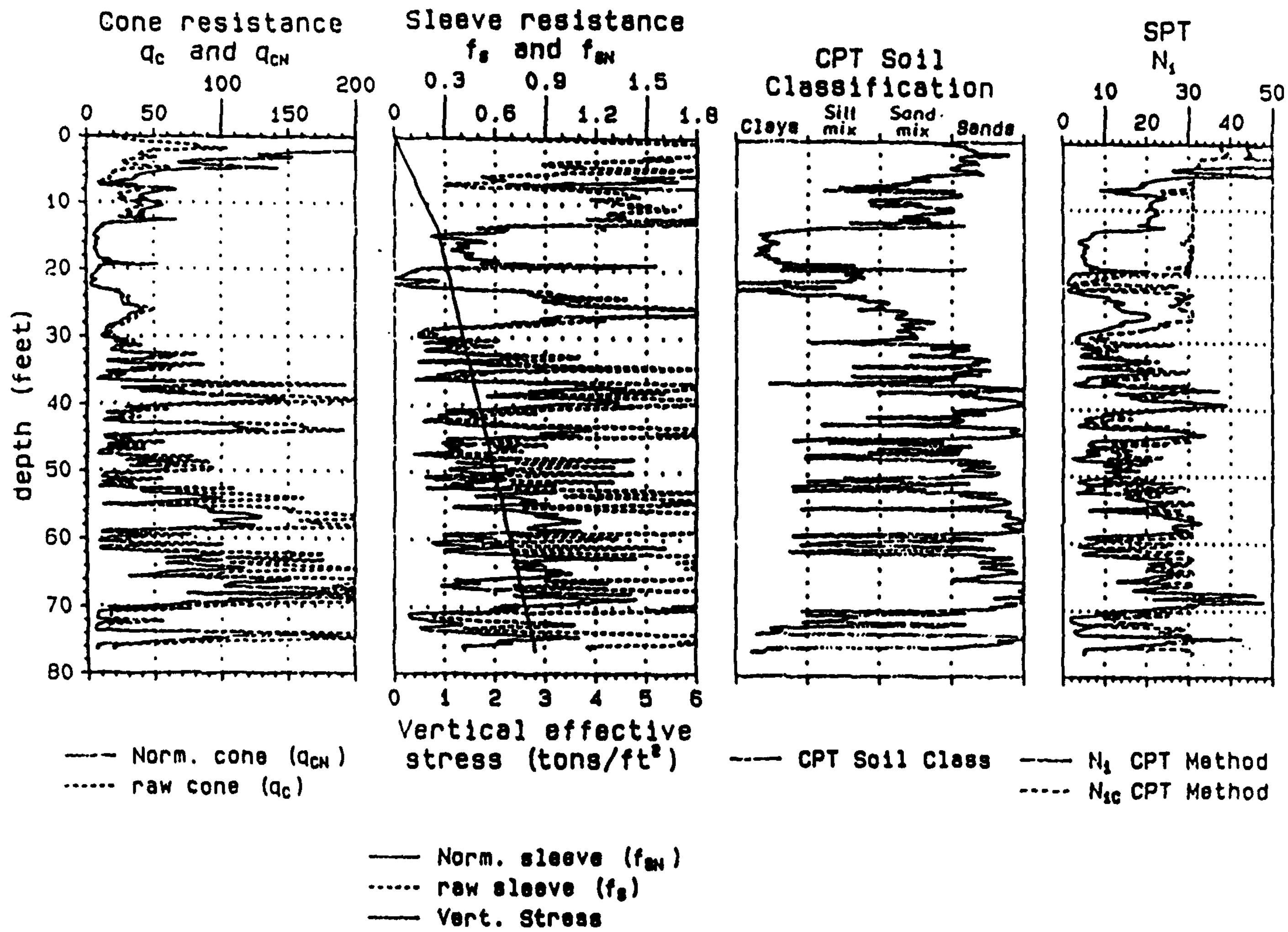
CPT Analysis

CPT-37
BARKLEY DAM



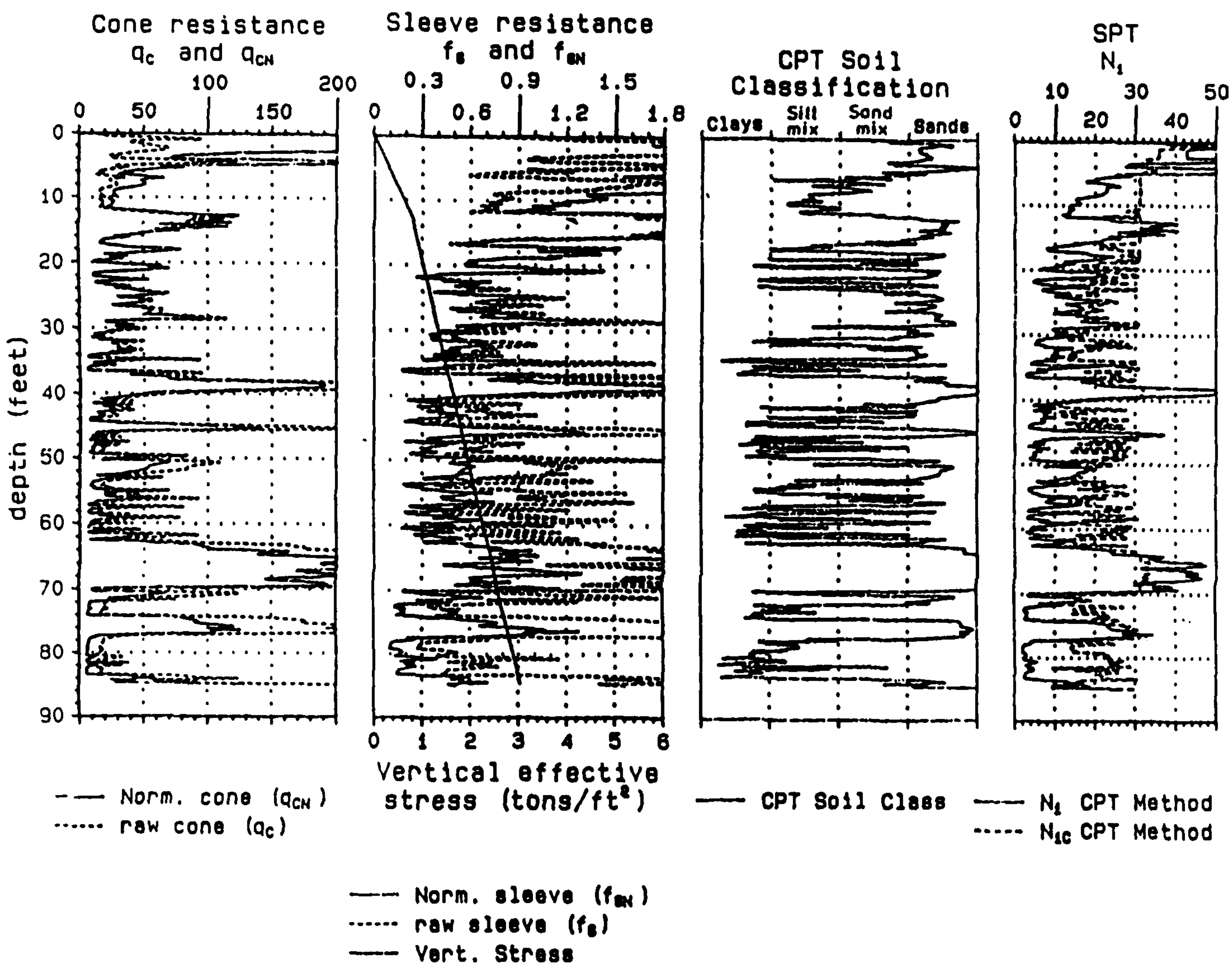
CPT Analysis

CPT-38
BARKLEY DAM



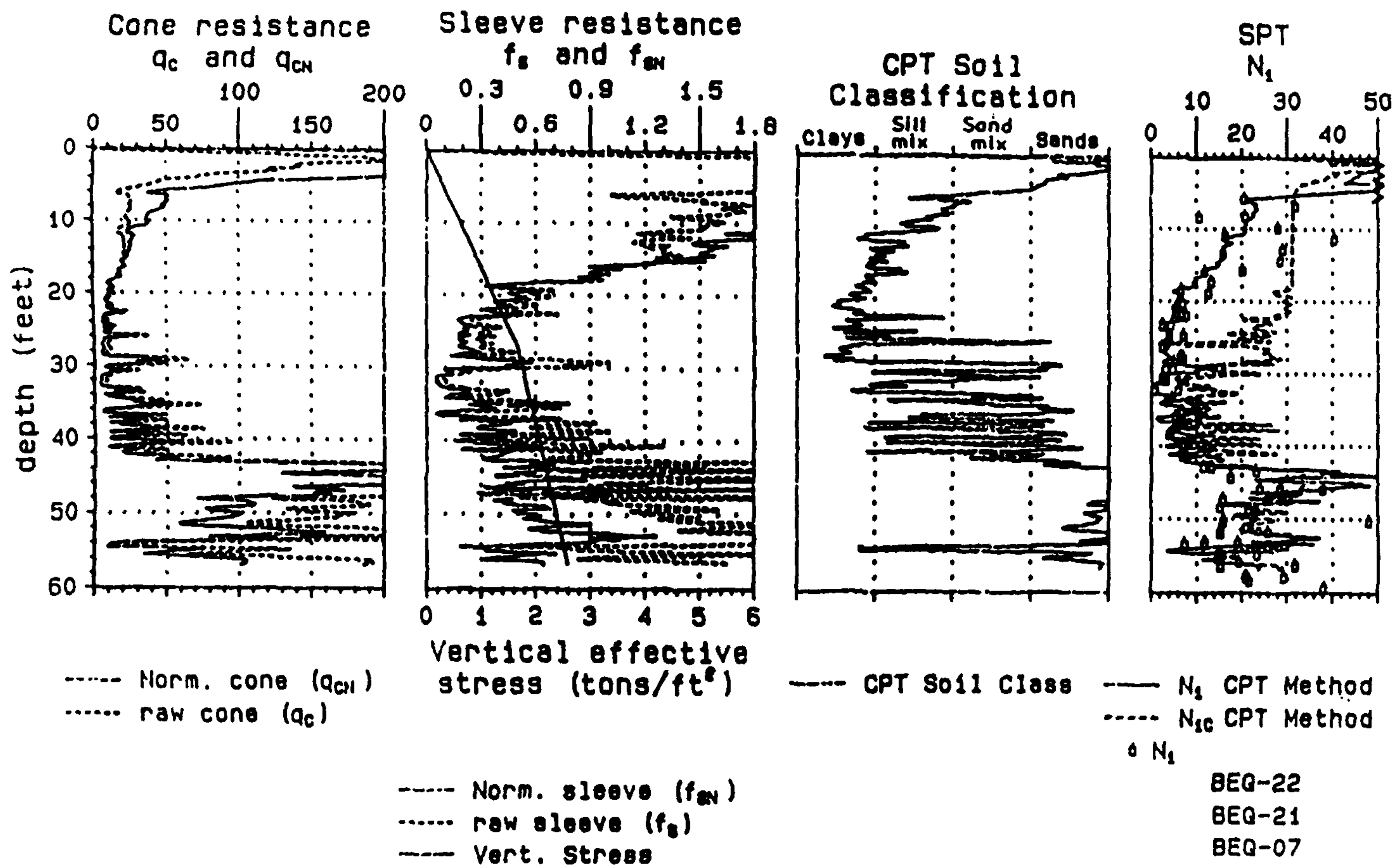
CPT Analysis

CPT-39
BARKLEY DAM



CPT Analysis

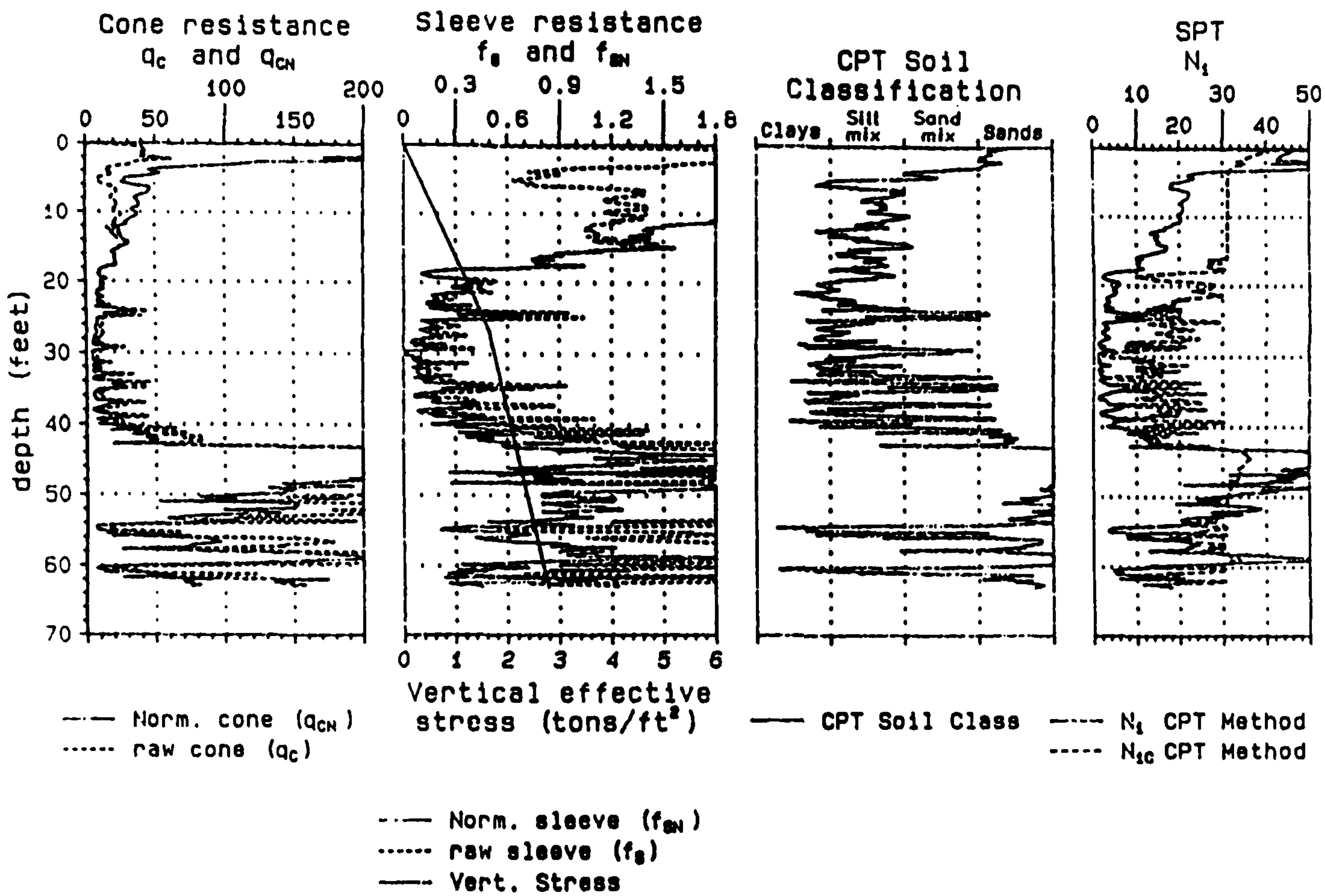
CPT-40
BARKLEY DAM



CPT Analysis

CPT-41

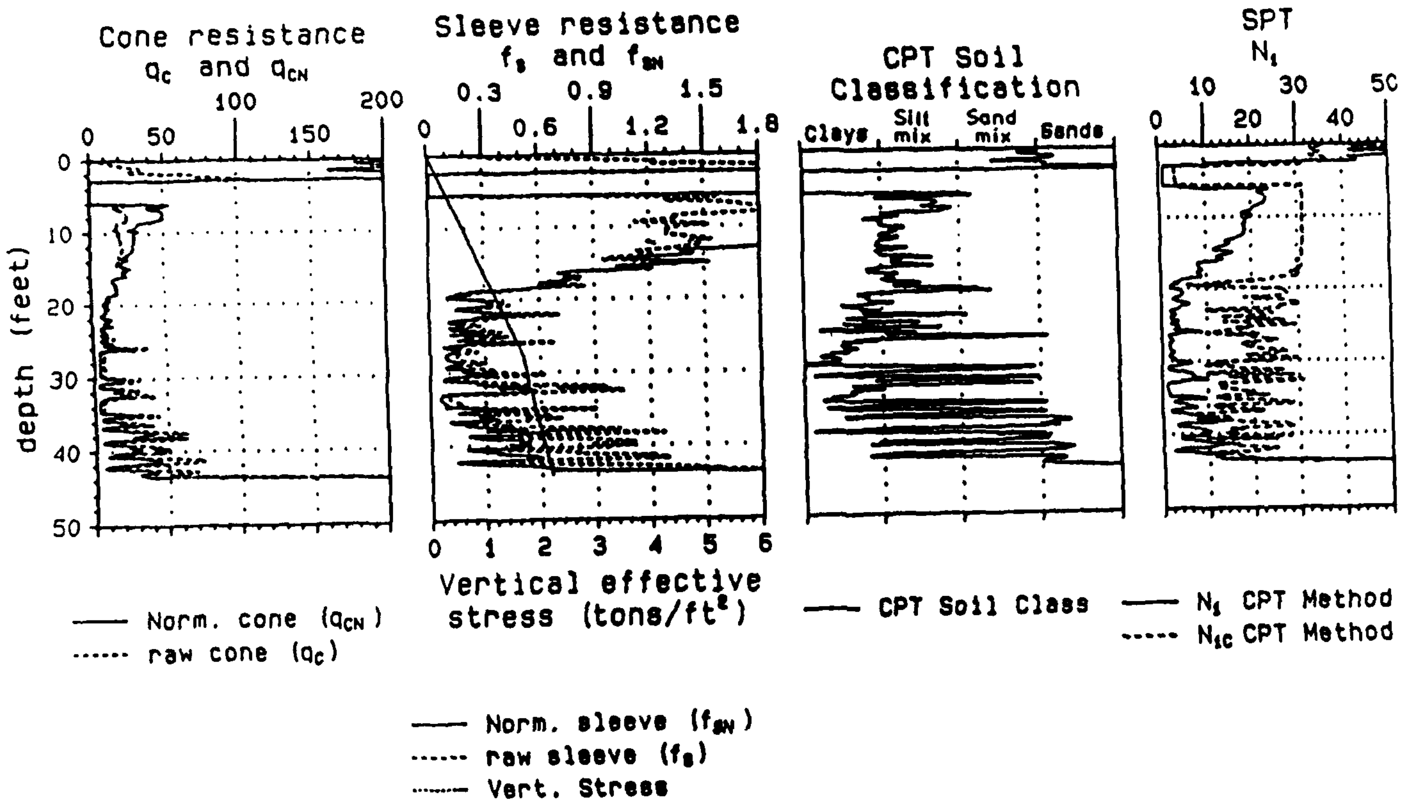
BARKLEY DAM



B42

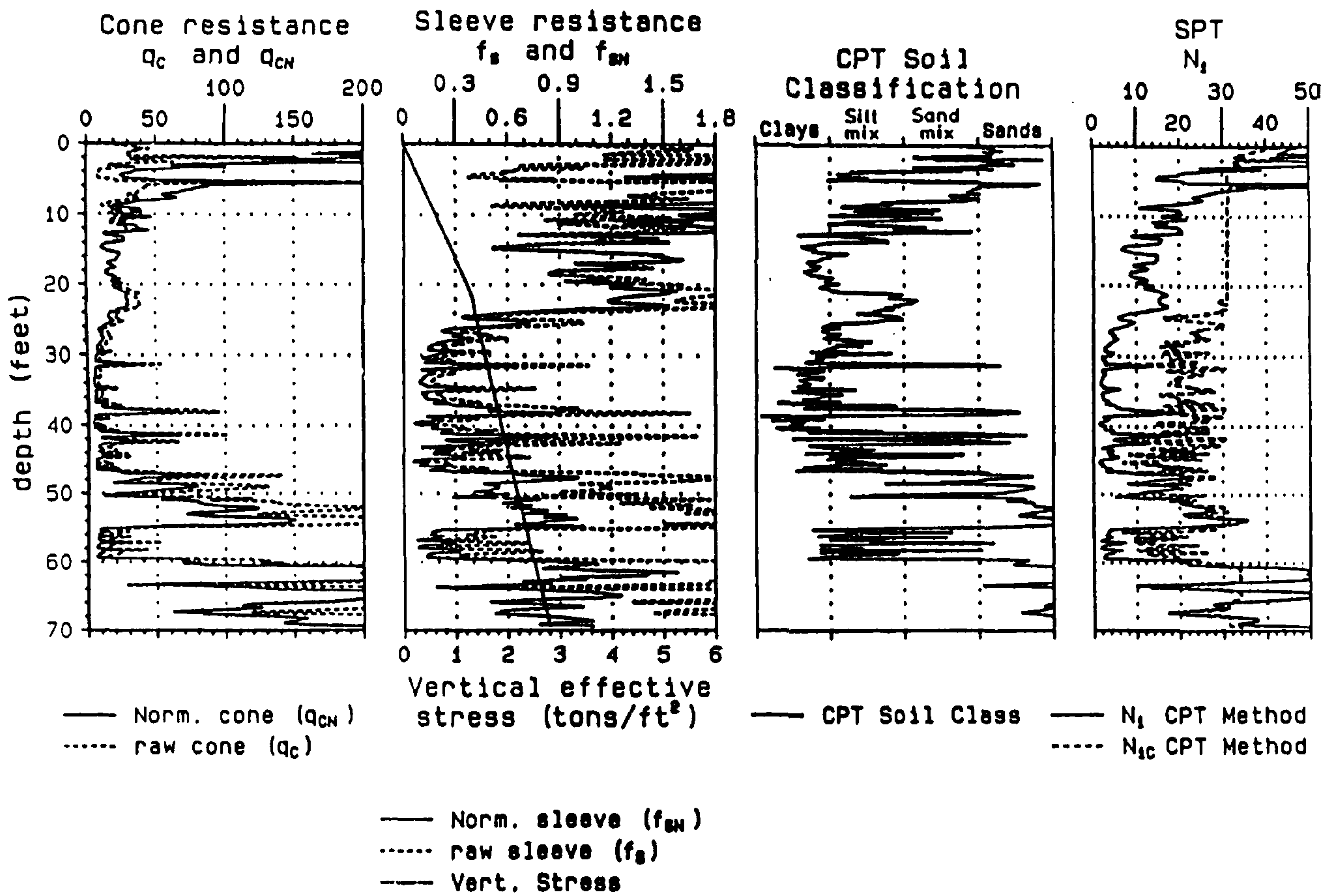
CPT Analysis

CPT-42
BARKLEY DAM



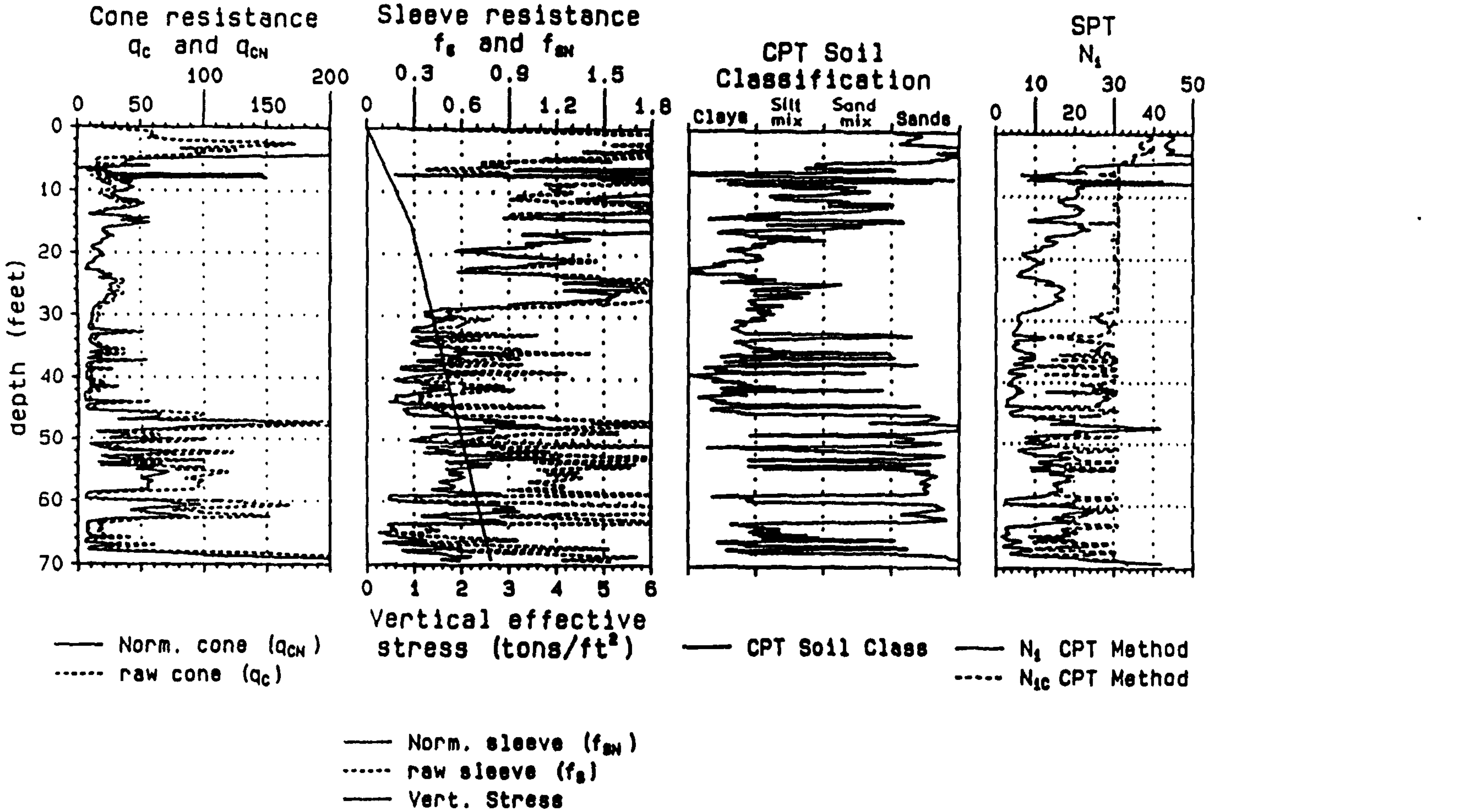
CPT Analysis

CPT-43
BARKLEY DAM



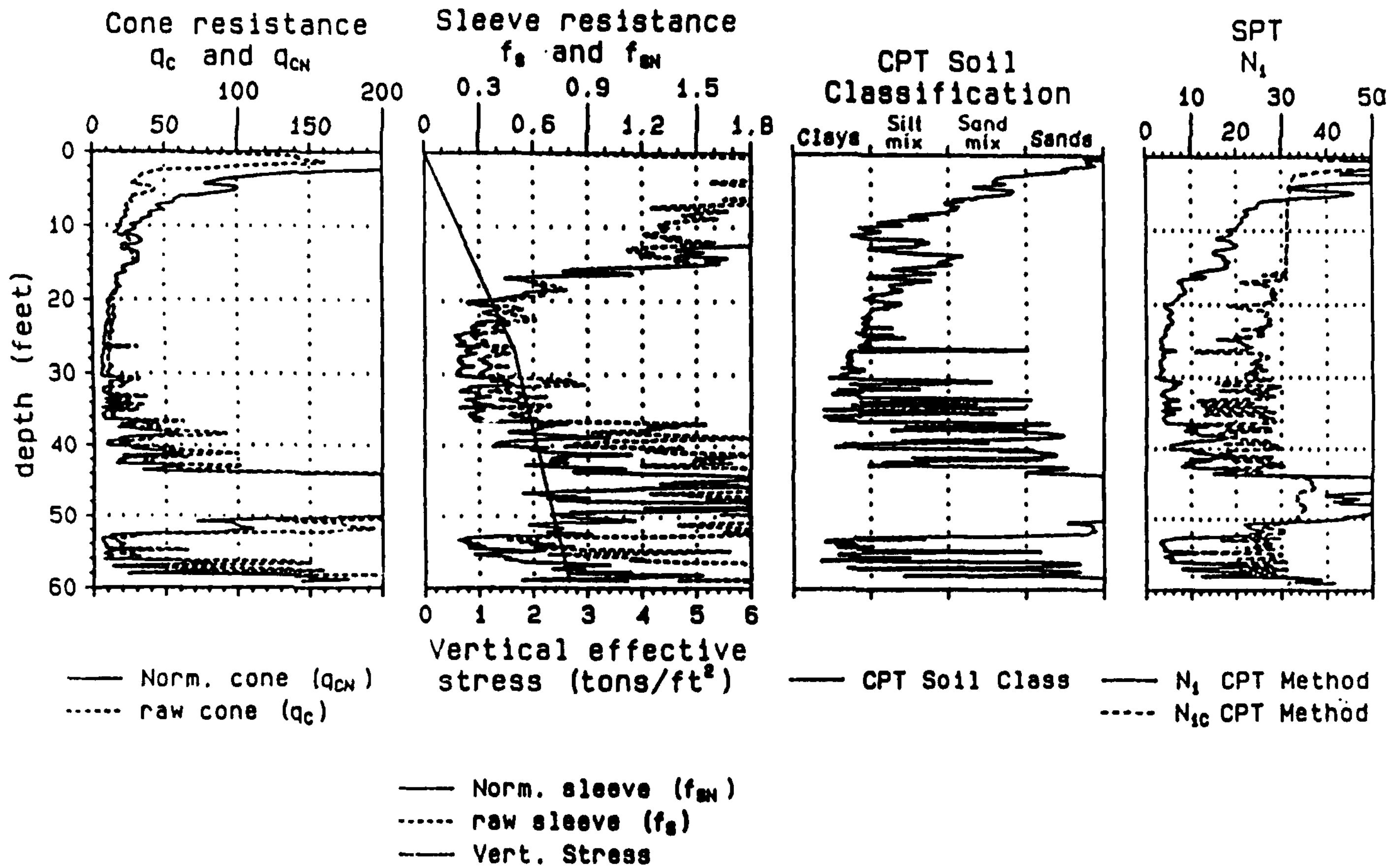
CPT Analysis

CPT-44
BARKLEY DAM



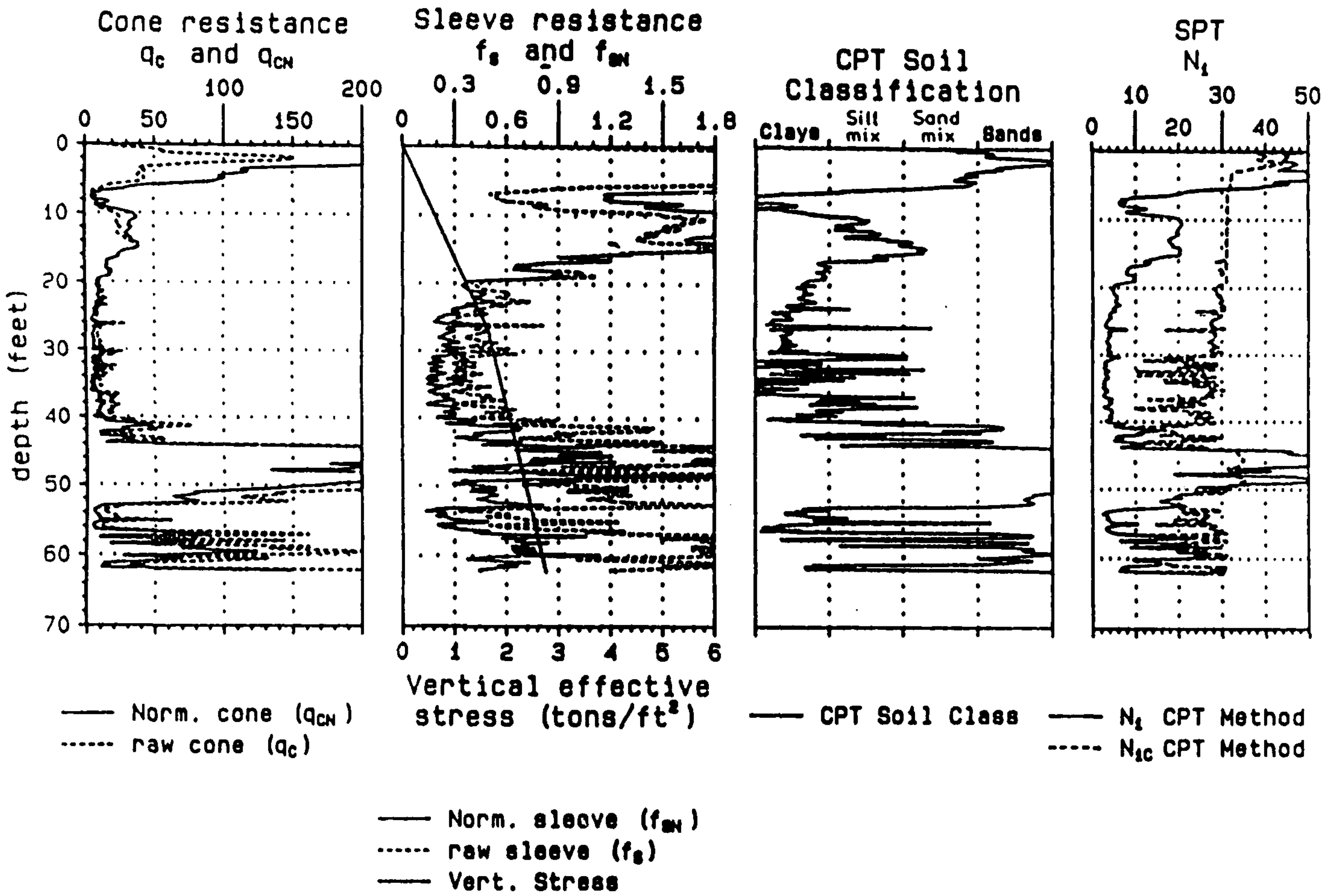
CPT Analysis

CPT-45
BARKLEY DAM



CPT Analysis

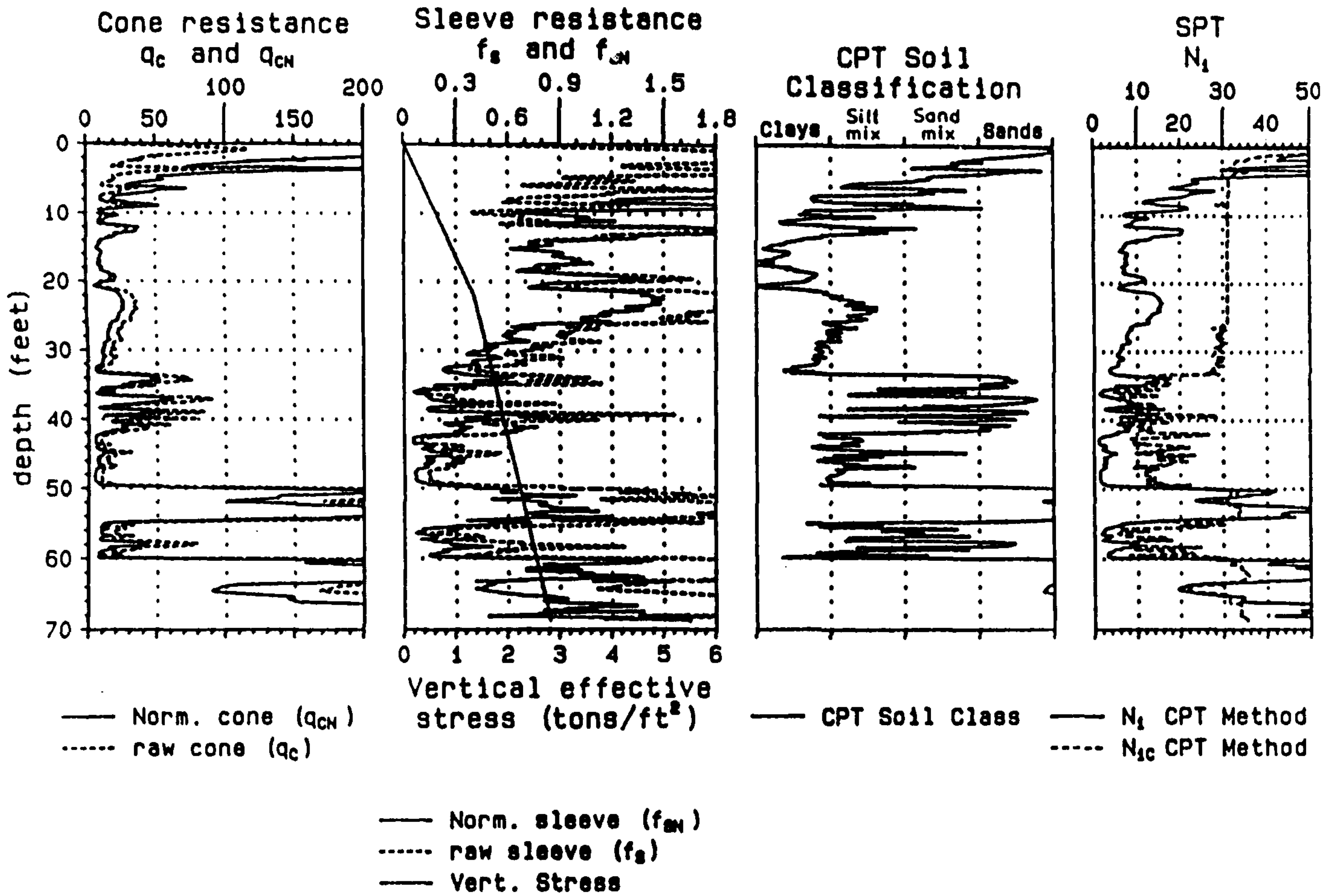
CPT-46
BARKLEY DAM



CPT Analysis

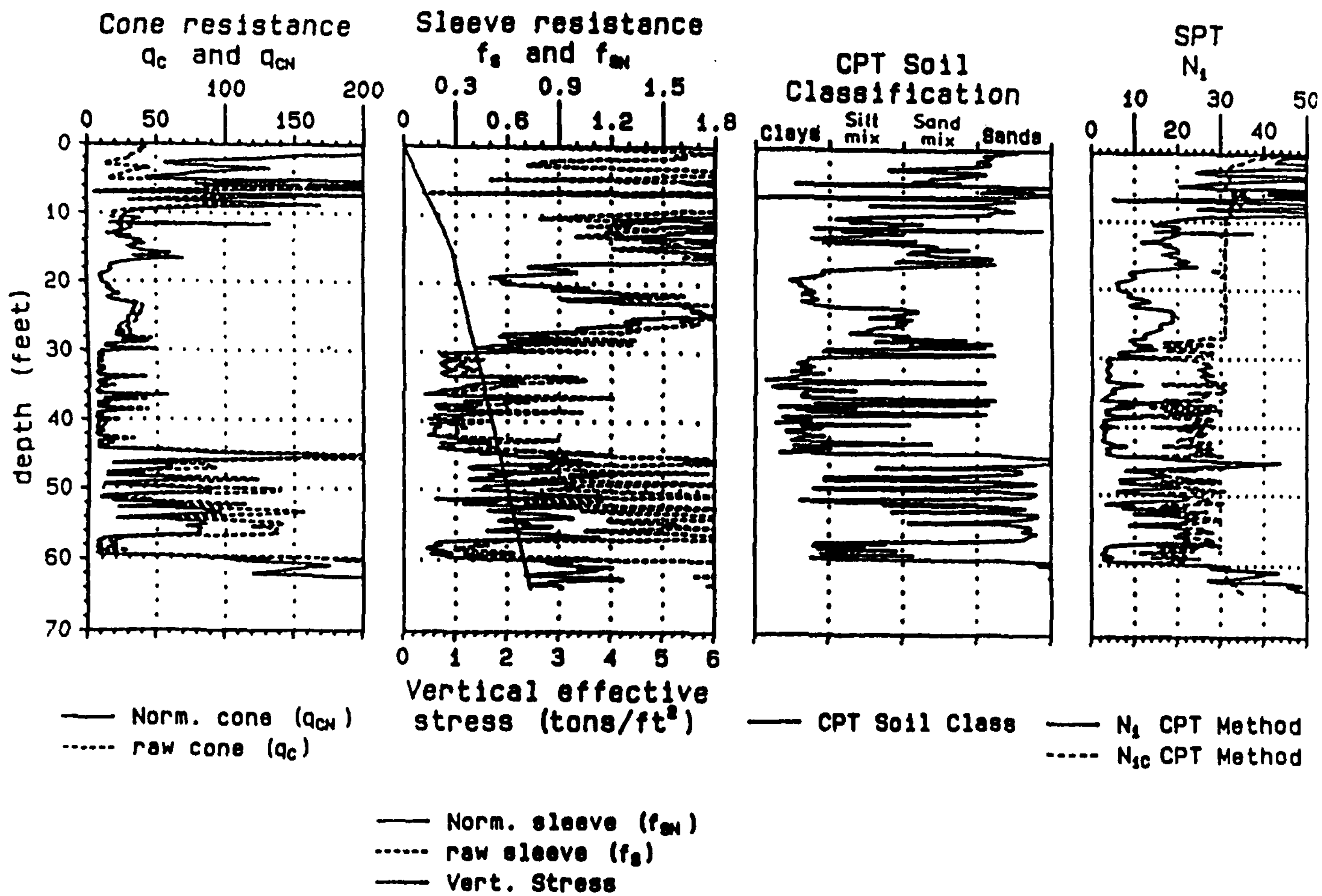
CPT-47

BARKLEY DAM



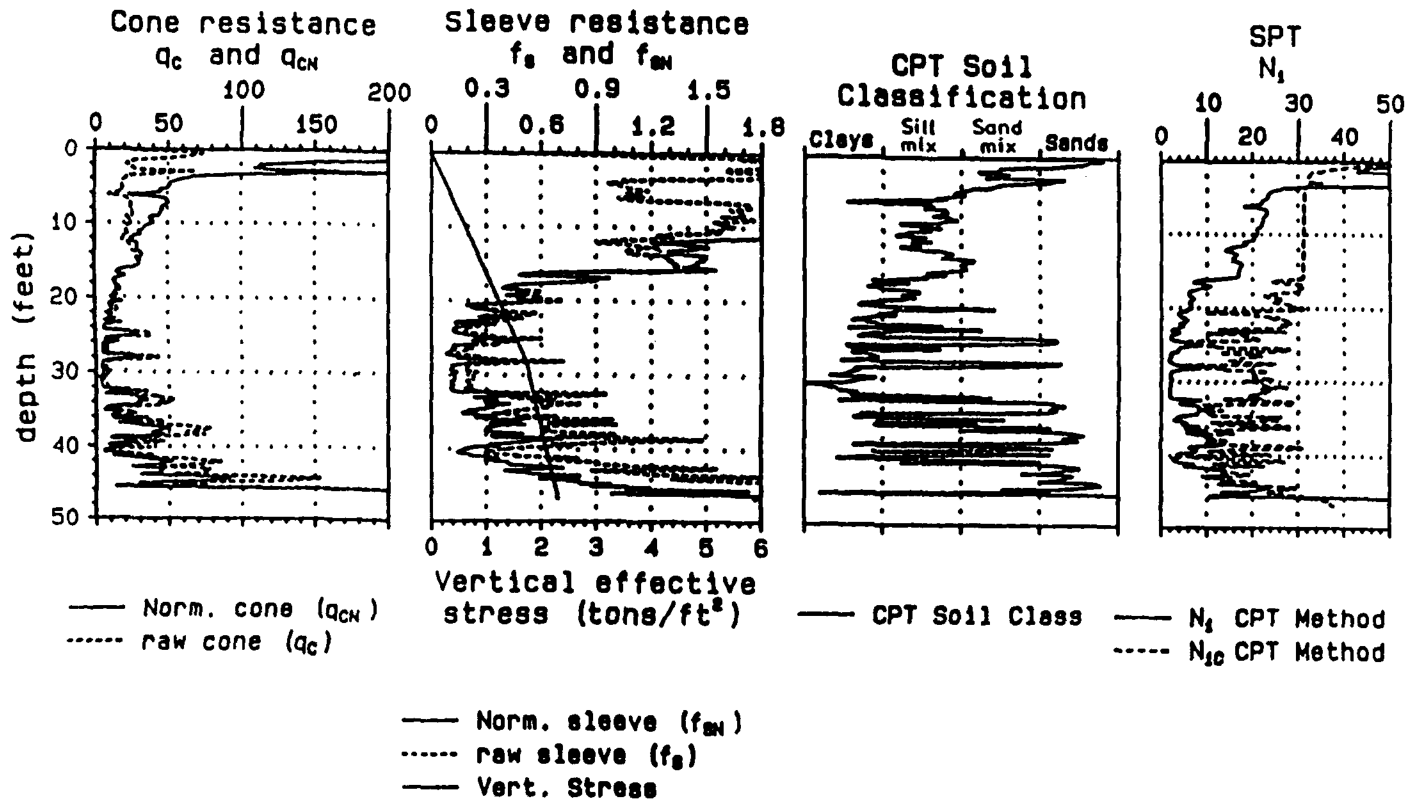
CPT Analysis

CPT-48
BARKLEY DAM



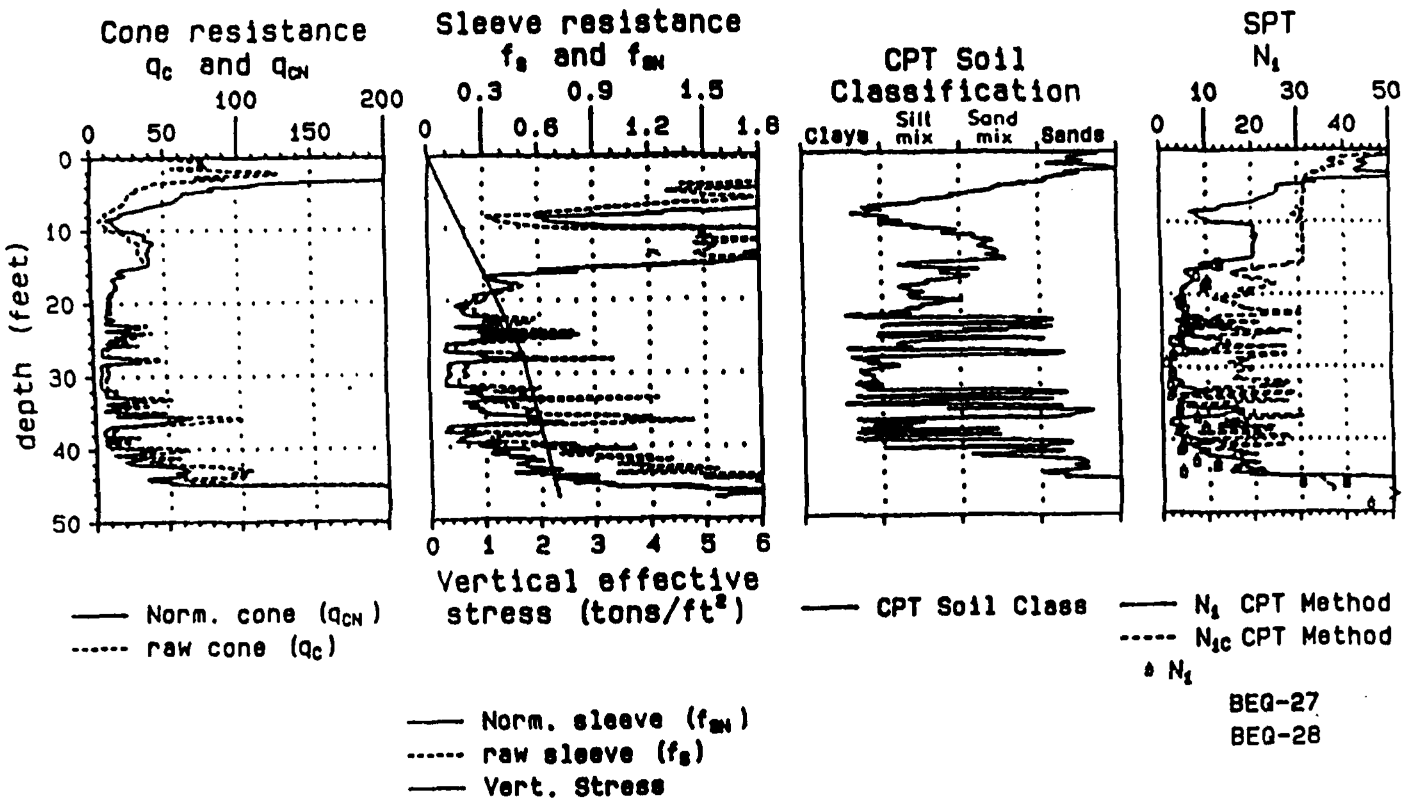
CPT Analysis

CPT-49
BARKLEY DAM



CPT Analysis

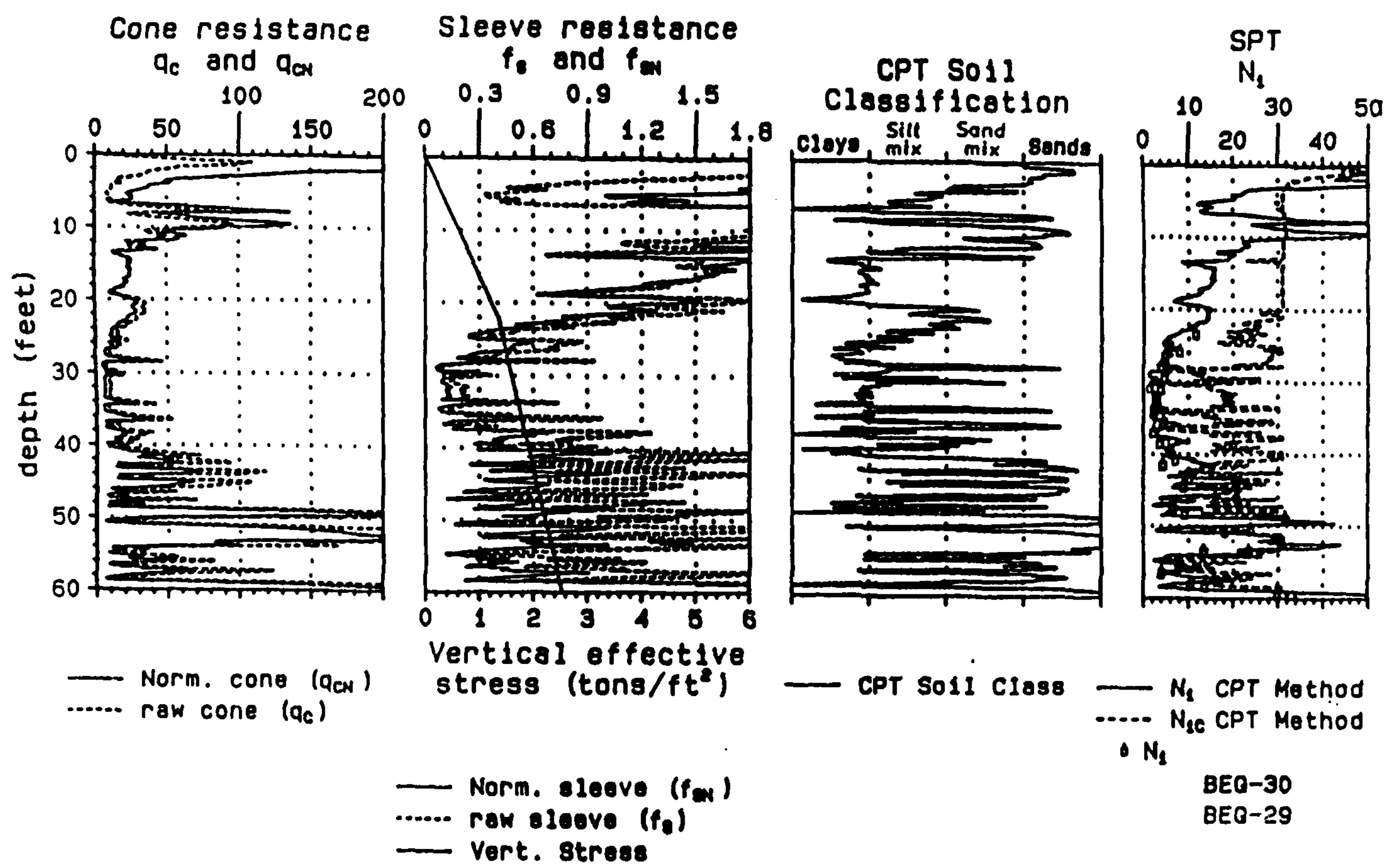
CPT-50
BARKLEY DAM



CPT Analysis

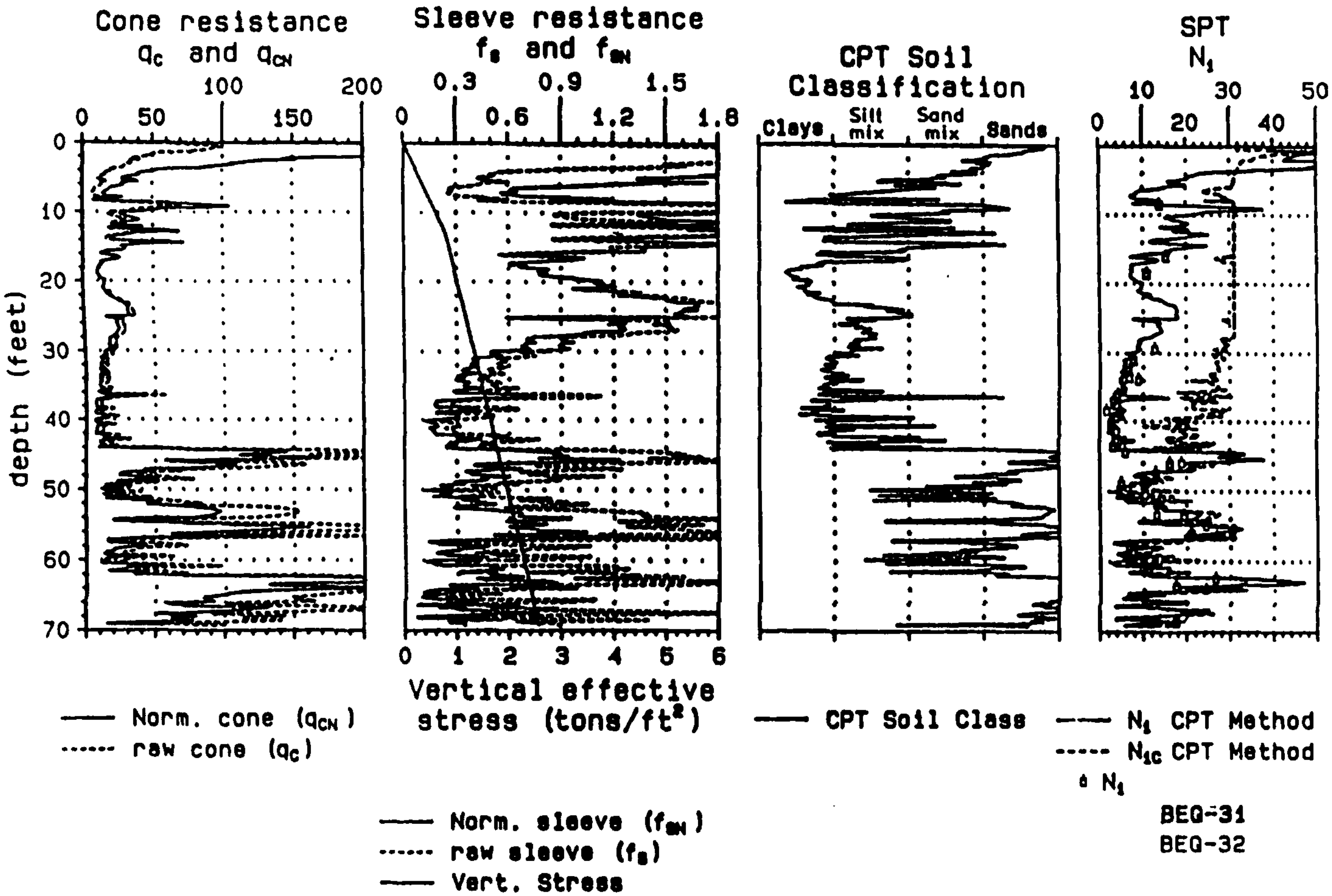
CPT-51
05/18/85

CONE-PORE LEAD : 0.0



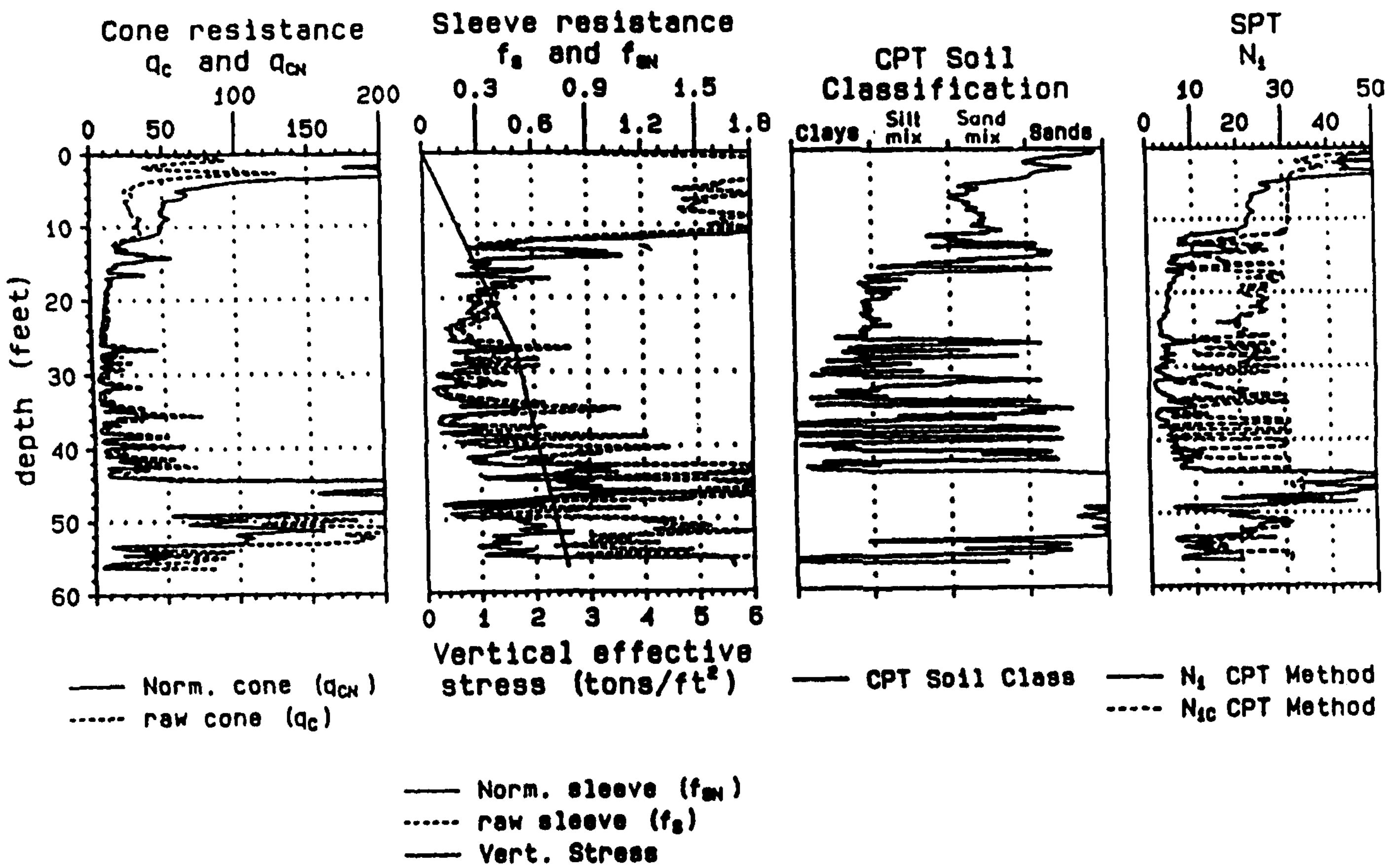
CPT Analysis

CPT-52
Barkley Dam



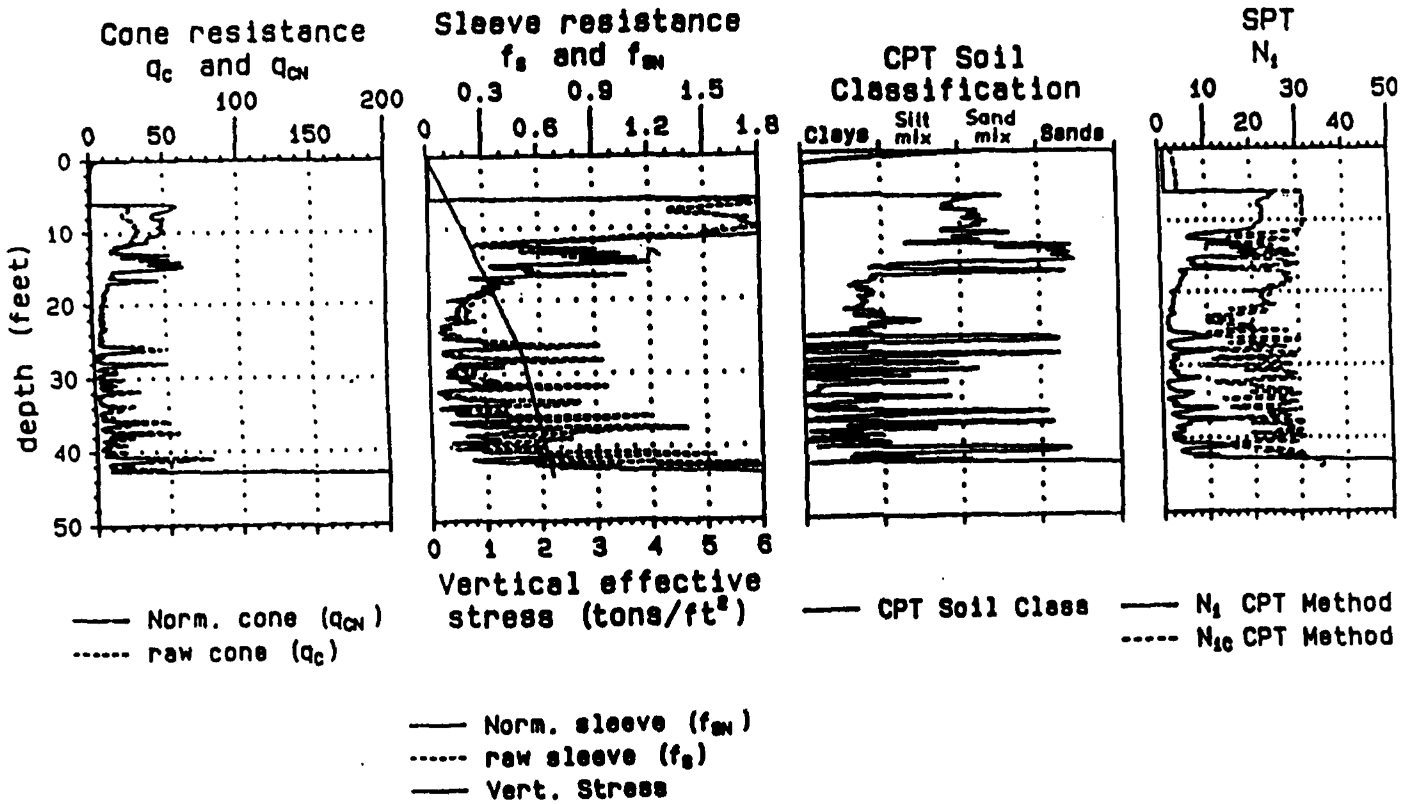
CPT Analysis

CPT-53
BARKLEY DAM



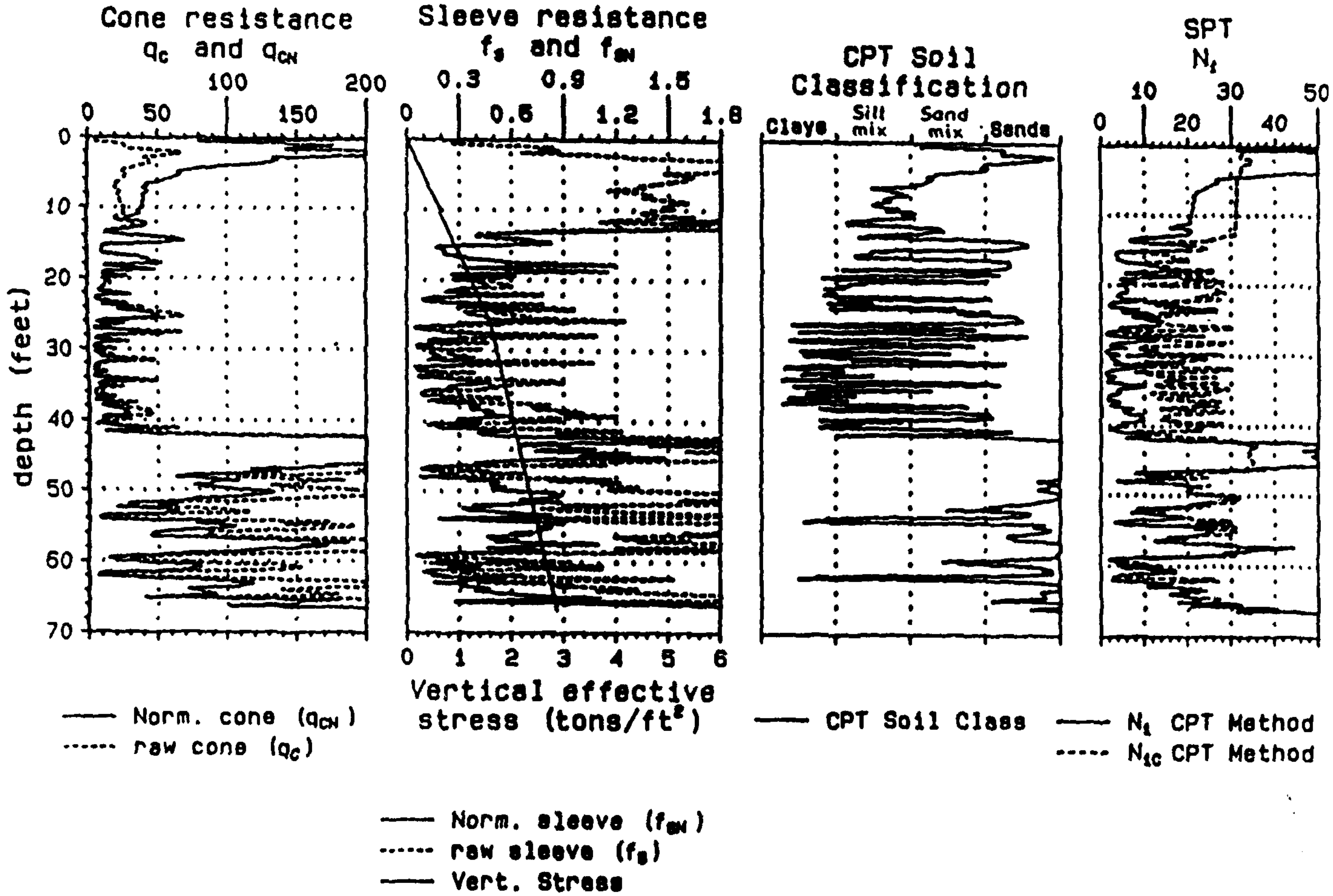
CPT Analysis

CPT-54
BARKLEY DAM



CPT Analysis

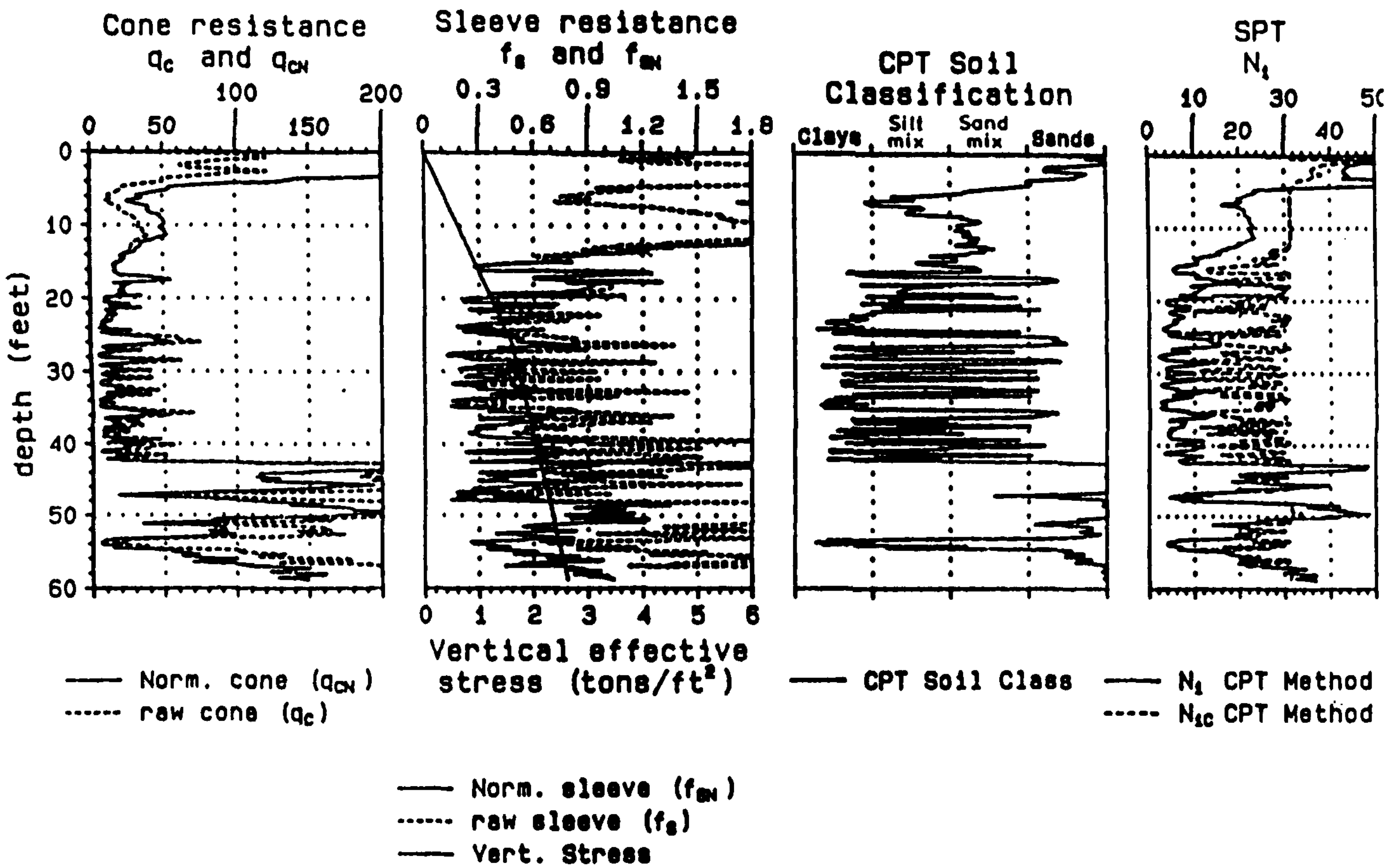
CPT-55
BARKLEY DAM



CPT Analysis

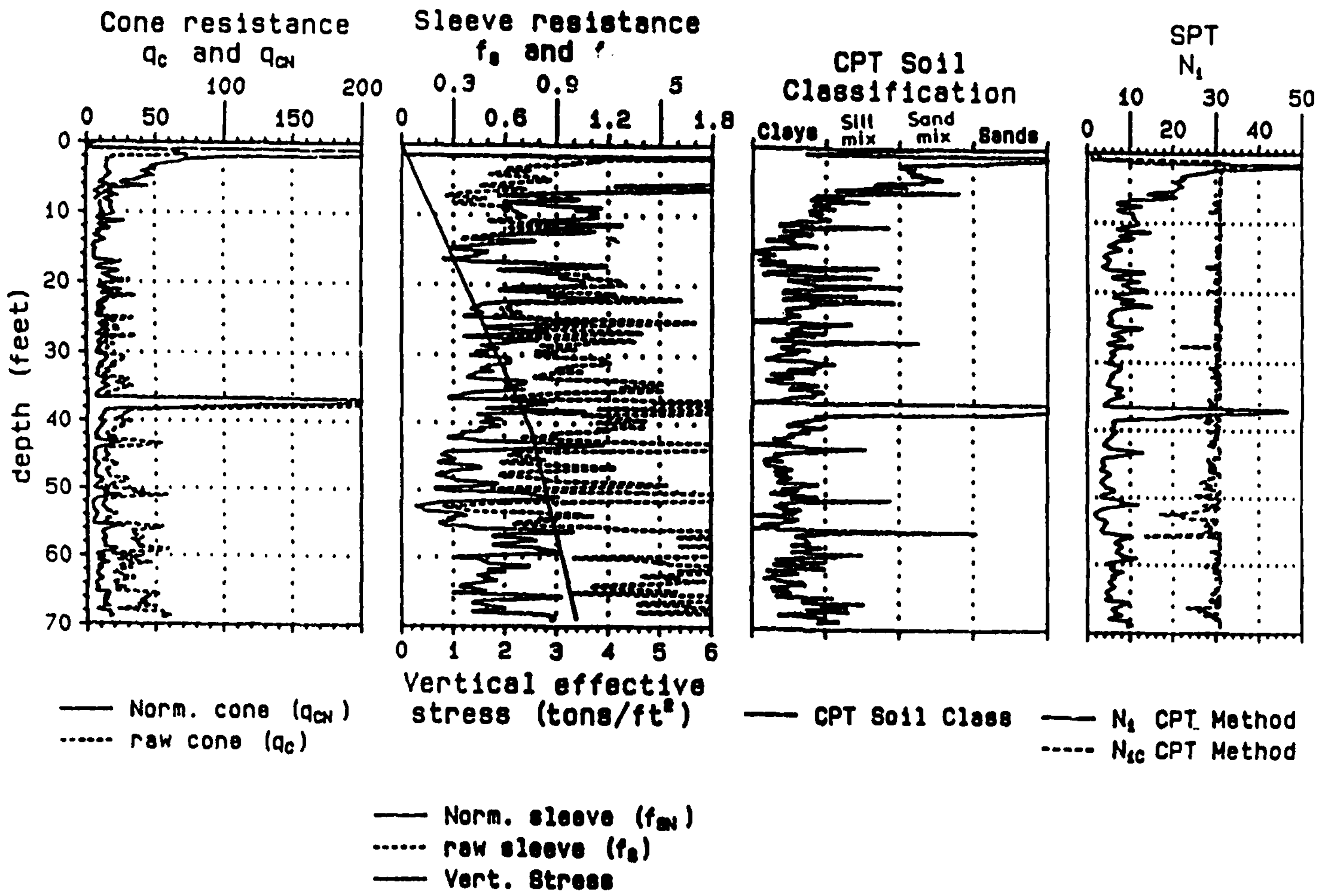
CPT-56

BARKLEY DAM



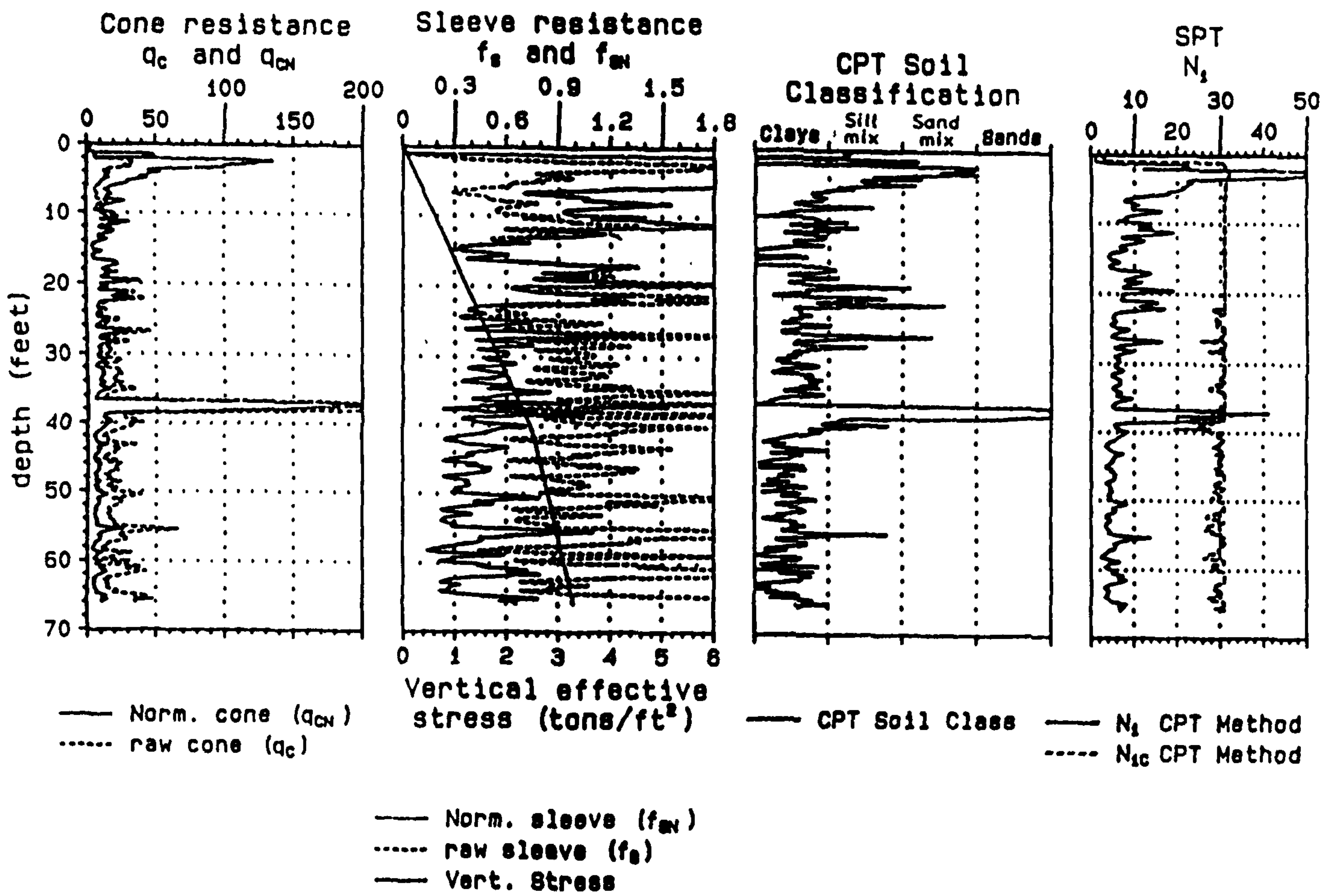
CPT Analysis

CPT-57
BARKLEY DAM



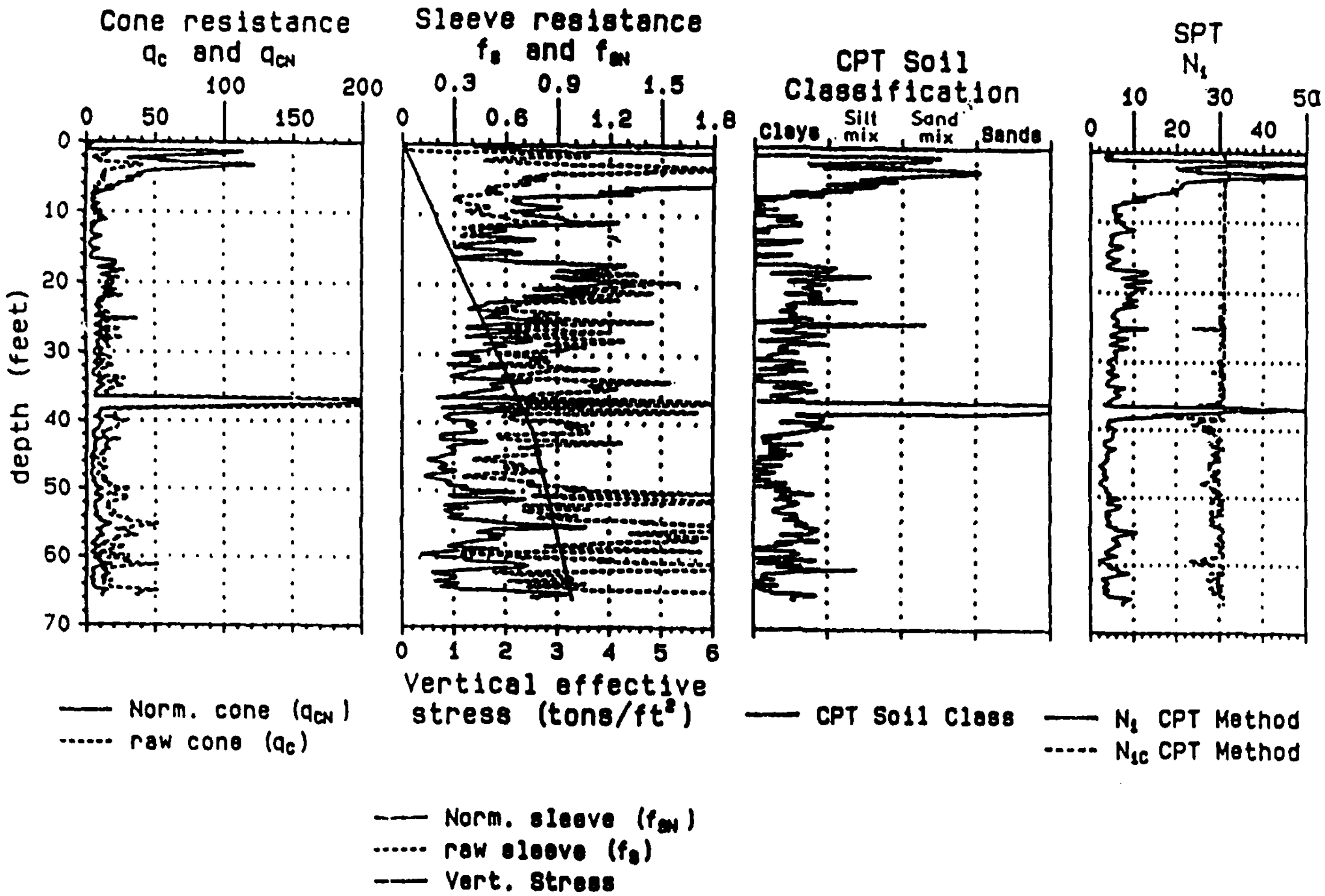
CPT Analysis

CPT-59
BARKLEY DAM



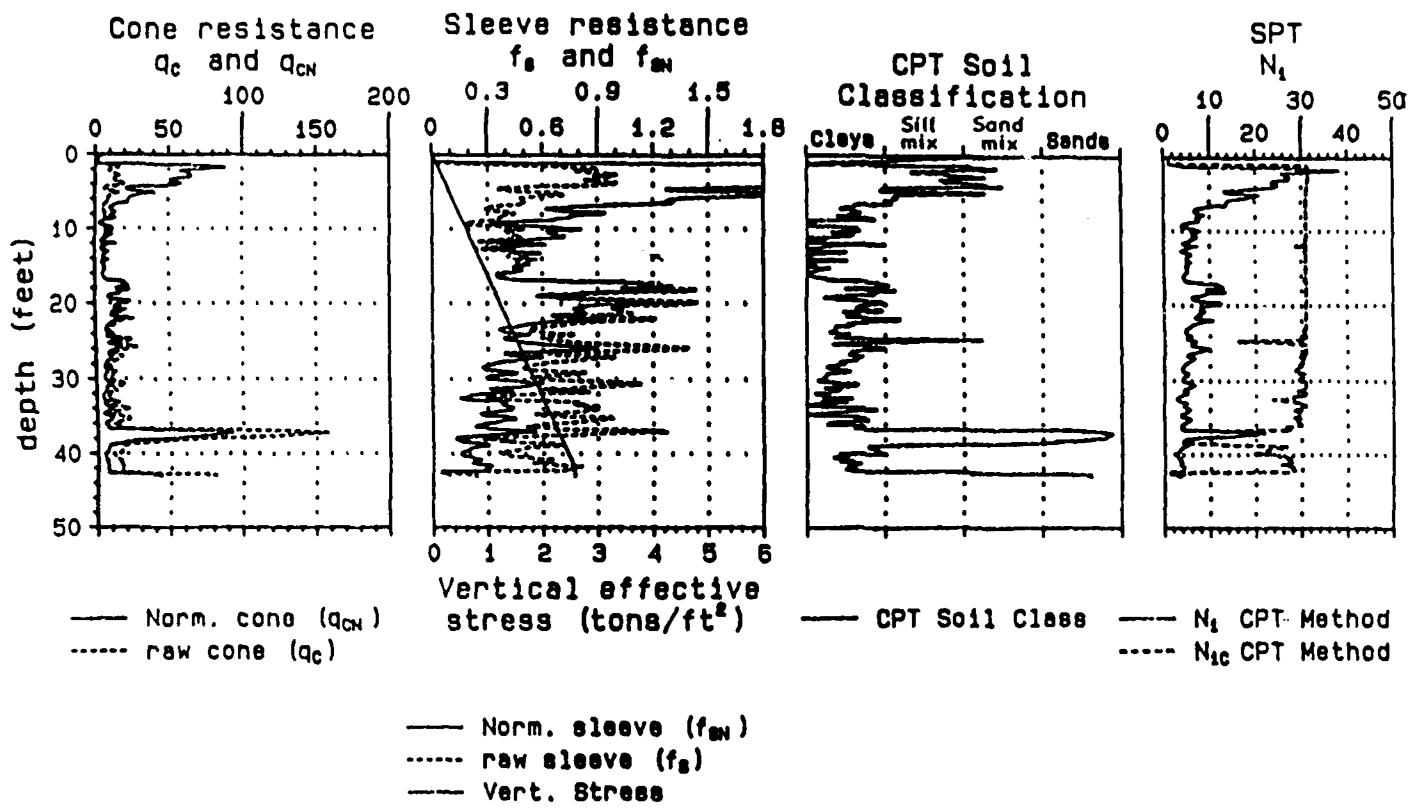
CPT Analysis

CPT-60
BARKLEY DAM



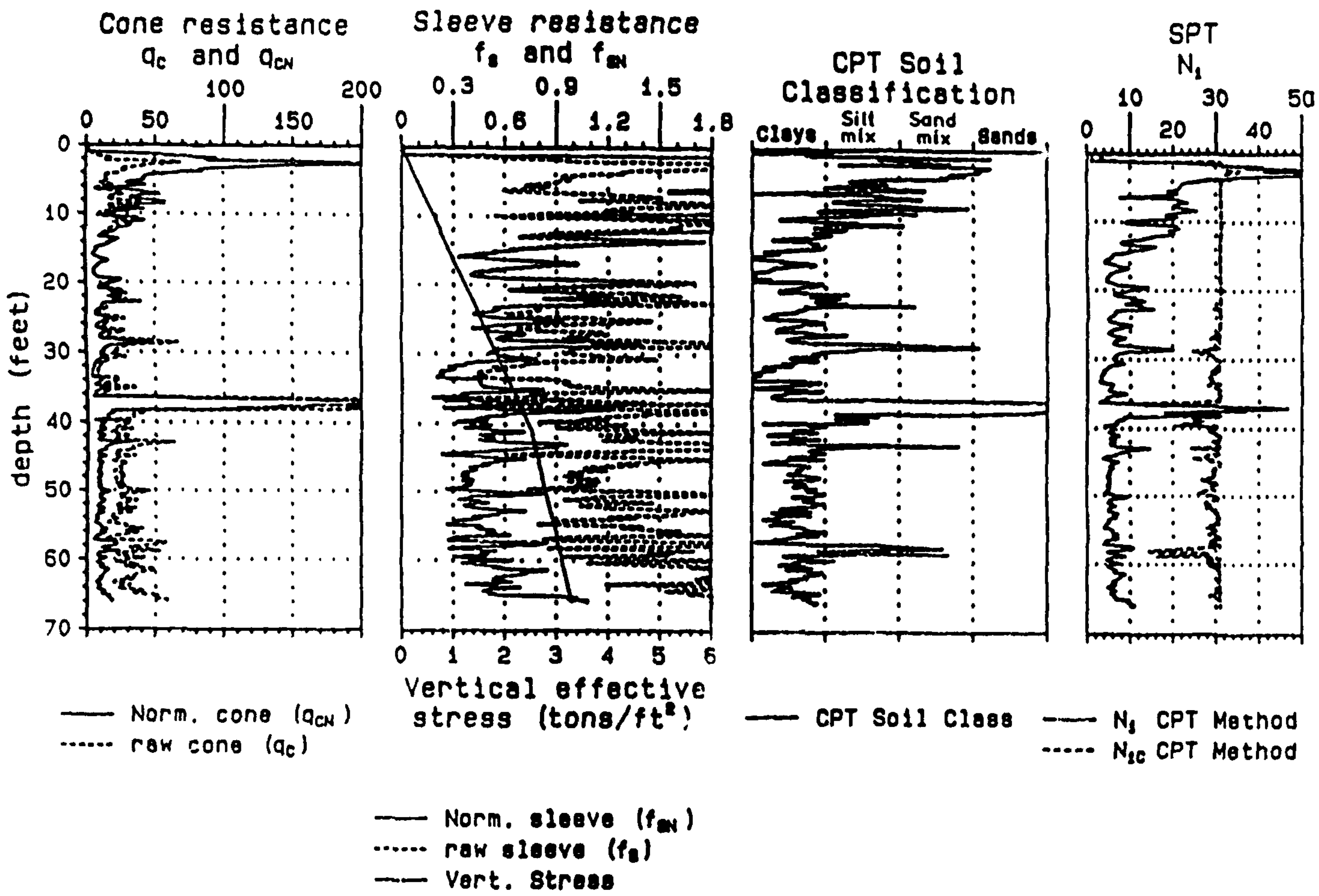
CPT Analysis

CPT-61
BARKLEY DAM



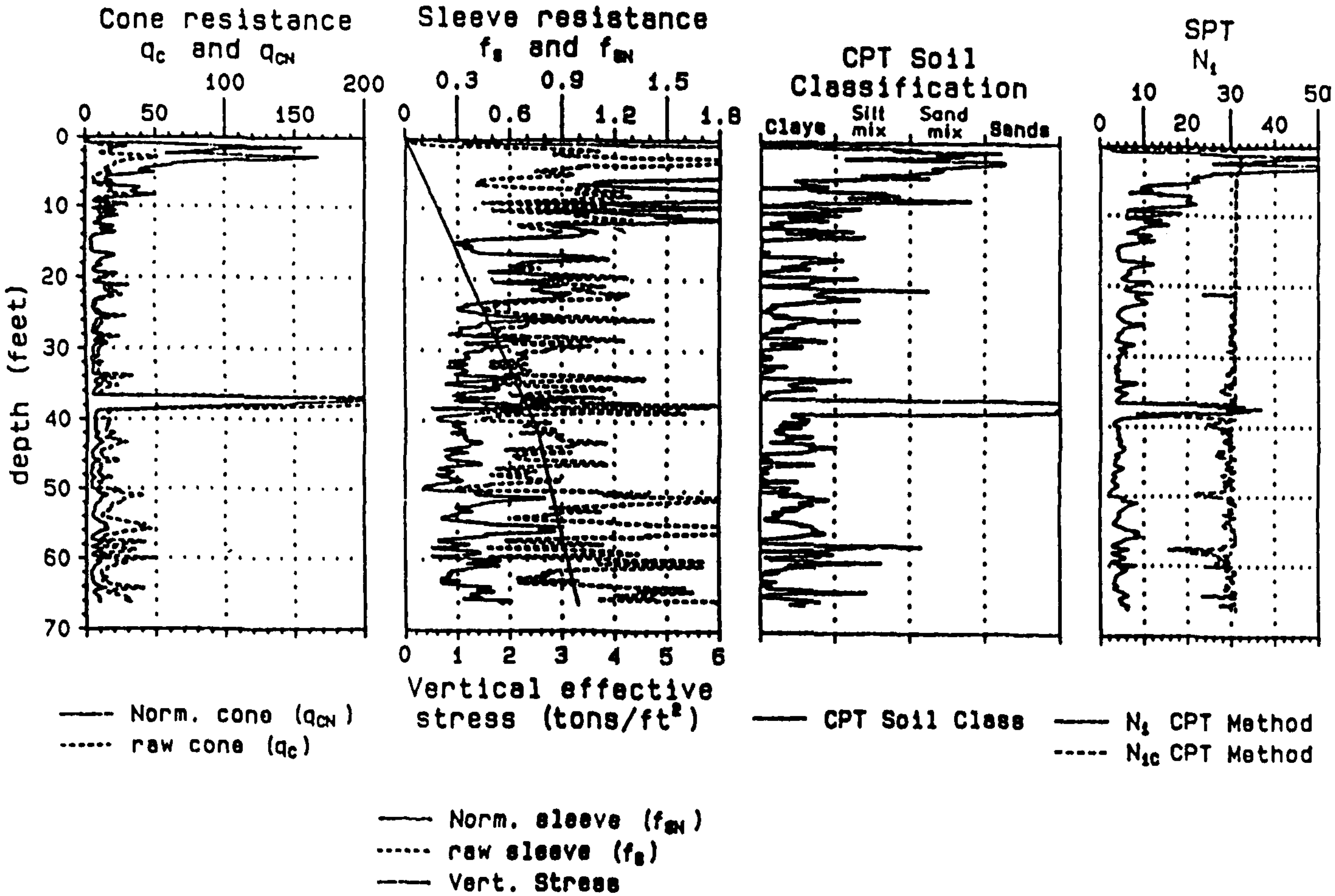
CPT Analysis

CPT-62
BARKLEY DAM



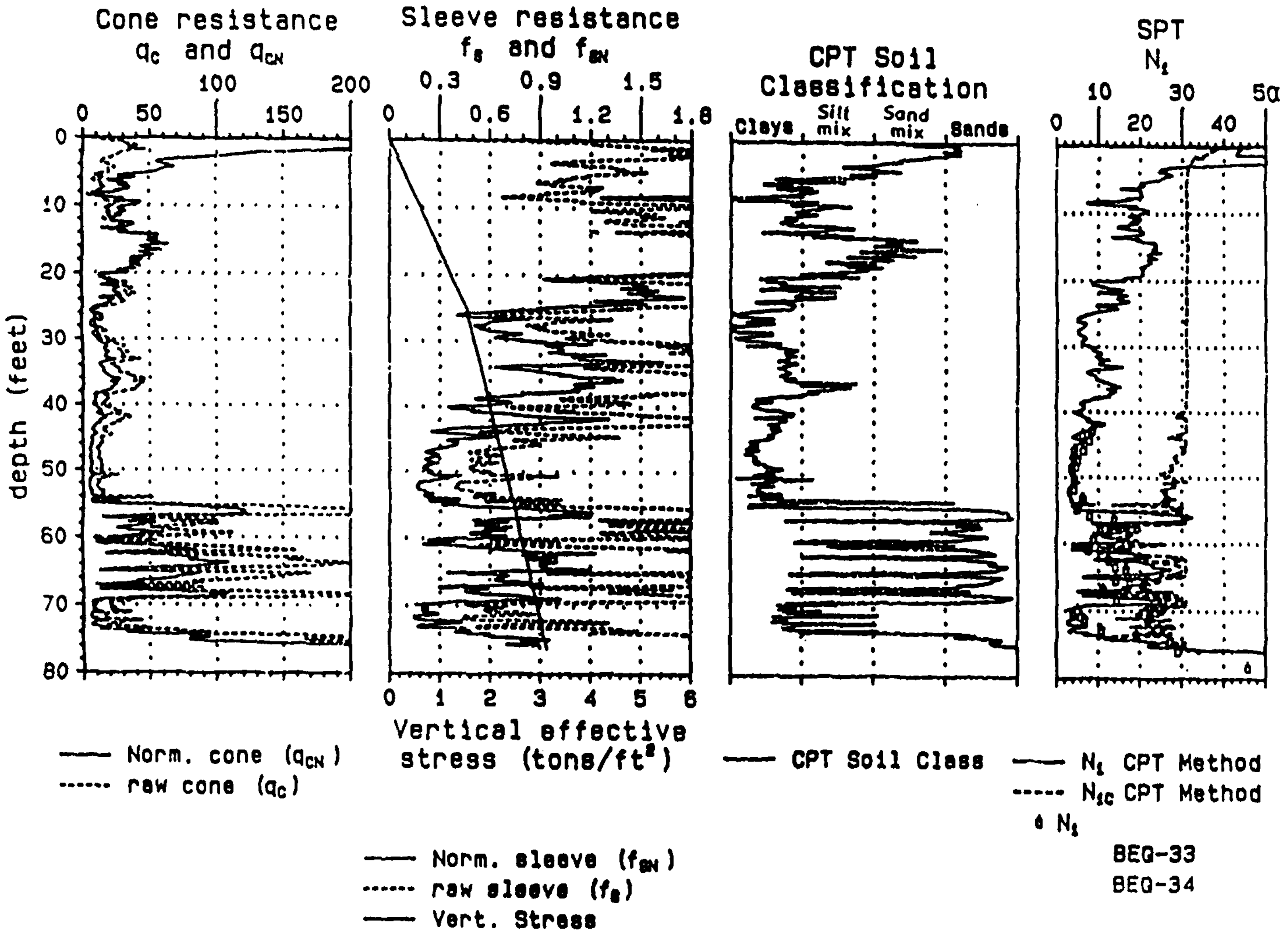
CPT Analysis

CPT-63
BARKLEY DAM



CPT Analysis

CPT-64
BARKLEY DAM



CPT Analysis

CPT-65
BARKLEY DAM

B64

APPENDIX C:

SPT LABORATORY DATA

BEC-01
Barkley dam

Depth feet	Elev feet	EVS tsf	Soil #	ID	(--%---) PL	Wn	LL	Wn/LL	(------D50------) L	Aver	H	(--P200--) L	Aver	H	(-----PDOS----) L	Aver	H	(SPT) N	N1	(-Nlc--) P200
1.0	353.6	0.0	1	C	17		30		0.02	0.07	0.20	15	57	73		25.5		7	44	
4.0	350.6	0.2	1	C	16		26			0.01			87		31.0	31.0	31.0	18	39	
7.1	347.5	0.4	1	C	16		32		0.01	0.11	0.25	5	52	86		38.0		25	40	
9.9	344.7	0.6	1	C	13		24			0.02			79			27.5		8	10	
13.1	341.5	0.8	1	S					0.14	0.18	0.25	5	13	18		7.0		15	17	21.3
16.0	338.6	0.9	1		15		22		0.05	0.10	0.20	15	45	62		19.0		9	10	16.3
19.1	335.5	1.0	1	S					0.10	0.14	0.25	5	26	32		13.0		10	10	15.9
25.0	329.6	1.1	1	C	17		26		0.02	0.10	0.20	15	49	77		19.5		5	5	10.9
28.1	326.5	1.2	1	S	20		20		0.10	0.18	0.25	5	22	39		10.0		22	20	27.2
31.0	323.6	1.3	1	C	18		25		0.03	0.08	0.20	15	55	72		20.5		5	4	
34.1	320.5	1.4	1	S	18		18		0.08	0.15	0.25	5	31	49		15.2		12	10	16.4
37.1	317.5	1.5	1	S					0.16	0.19	0.25	5	13	18		7.2		26	21	25.9
40.0	314.6	1.6	1	S					0.09	0.14	0.25	5	30	43		15.5		10	8	13.9
43.5	311.1	1.7	1	C	17		22		0.06	0.10	0.20	15	41	52		15.0		5	4	10.0
46.1	308.5	1.8	1	C	19		28		0.06	0.12	0.25	5	42	58		14.5		14	10	17.3
49.0	305.6	1.9	1	C	18		35		0.05	0.10	0.20	15	47	60		20.5		7	5	
52.1	302.5	2.0	1	S	16		28		0.08	0.15	0.25	5	31	48		16.0		10	7	13.0
55.2	299.4	2.1	1	C	18		35		0.02	0.14	0.25	5	35	69		25.0		10	7	
58.0	296.6	2.1	1	C	16		28			0.03			67			23.0		6	4	
61.1	293.5	2.3	1	S	17		29			0.09			43			12.5		13	9	15.1
64.1	290.5	2.3	1	S						0.38			8			2.0		32	21	22.7
67.0	287.6	2.4	1							0.35			7			2.0		44	28	30.3
70.1	284.5	2.5	1	S	16		32			0.83			28			11.0		33	21	30.0
73.2	281.4	2.6	1	S						0.21			19			7.5		35	22	28.9
76.0	278.6	2.7	1	S	13		20			0.26			13			5.0		48	29	32.5
79.0	275.6	2.8	1	S	14		16			0.27			19			6.8		31	19	24.8
85.1	269.5	3.0	1	S					0.17	0.17	0.17		13			3.8		84	49	32.5
110.2	244.4	3.8	1	S	14		26			0.17			16			6.9		45	23	29.2

BCG-02

Barkley dam

Depth feet	Elev feet	EVS tsf	#	Soil ID	(--%---) PL	Wn	LL	Wn/LL	(------D50------) L	Aver	H	(--P200--) L	Aver	H	(-----P0G5-----) L	Aver	H	(SPT) N	N1	(-N1c--) P200
73.1	274.1	2.3	1	S						0.29			9			3.0		54	36	31.2
76.2	271.0	2.4	1	S						0.17			12			5.5		33	21	25.5
82.1	265.1	2.6	1	S						0.83			7			2.1		21	13	13.9
91.0	256.2	2.8	1	S						0.27			9			3.1		59	35	31.8
97.1	250.1	3.0	1	S						0.21			18			4.7		26	15	20.4
100.2	247.0	3.1	1	S						0.23			8			3.0		49	28	30.4
103.1	244.1	3.2	1	S	13		19			0.20			30			9.0		42	23	30.7
106.0	241.2	3.3	1	S						0.26			15			6.0		30	17	21.7

ECQ-03

Barkley dam

Depth feet	Elev feet	EVS tsf	#	Soil ID	(--%---) PL	Wn	LL	Wn/LL	(------D50 --L	Aver	H--)	(--P200-- L	Aver	H)	(-----PC05 --L	Aver	H-)	(SPT) N	N1	(-N1c--) P200
1.0	348.6	0.0	1	C	21		28		0.01	0.08	0.25	5	63	84		28.0		14	>99	
4.1	345.5	0.2	1	C	18		36		0.01	0.07	0.20	15	66	92		42.0		7	15	
10.1	339.5	0.4	1	S			17		0.07	0.12	0.25	5	39	50		13.8		12	18	26.9
13.2	336.4	0.5	1	C	18		23		0.02	0.09	0.20	15	55	82		28.0		5	7	
16.1	333.5	0.6	1	C	18		28		0.01	0.06	0.20	15	66	85		32.0		3	4	
19.0	330.6	0.7	1	S			18		0.10	0.17	0.25	5	19	32		12.0		11	13	18.4
22.1	327.5	0.8	1	M	18		19		0.05	0.09	0.20	15	51	64		21.0		5	6	
25.0	324.6	0.9	1	C	21		28		0.02	0.08	0.20	15	64	89		29.0		1	1	
28.1	321.5	1.0	1	C	20		26		0.02	0.08	0.20	15	63	87		26.0		1	1	
31.2	318.4	1.1	1	C	22		31		0.02	0.08	0.20	15	61	88		26.0		0	0	
34.1	315.5	1.2	1	S	18		18		0.08	0.14	0.25	5	32	45		14.5		18	17	24.8
37.0	312.6	1.3	1	C	18		24		0.04	0.10	0.20	15	44	61		19.0		6	5	11.6
43.1	306.5	1.4	1	S			19		0.09	0.16	0.25	5	27	41		12.5		10	8	13.8
46.2	303.4	1.5	1	S			16		0.21	0.22	0.25	5	18	24		9.0		16	13	17.8
49.1	300.5	1.6	1	S			17		0.14	0.19	0.25	5	17	27		9.5		17	13	18.2
52.2	297.4	1.7	1	S			19			0.22			10			4.3		26	20	22.7
55.0	294.6	1.8	1	S			18			0.19			13			5.5		32	24	29.0
58.0	291.6	1.9	1	S			19			0.20			17			9.3		19	14	18.8
61.2	288.4	2.0	1	S			17			0.23			15			5.1		43	30	32.5
64.1	285.5	2.1	1	S			18			0.24			8			4.0		36	25	28.6
70.0	279.6	2.3	1	S			18			0.23			10			4.5		39	26	30.2
76.2	273.4	2.5	1	C	21		30			0.06			55			25.0		7	4	
79.0	270.6	2.6	1	S			16			0.26			8			3.5		37	23	25.5
85.0	264.6	2.7	1	S			16			0.19			19			7.5		30	18	24.3
88.1	261.5	2.8	1	C	16		26			0.02			73			29.0		8	5	
91.0	258.6	2.9	1	C	18		30			0.02			78			31.0		11	6	
94.1	255.5	3.0	1	C	17		29			0.02		77	76	77		30.5		7	4	

BEQ-94
Barkley dam

Depth	Elev	CUS	Soil		(--%---		---)		(------D50	------	------)	(--P200	--	--)	(-----P005	----	----)	(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver	---H--)	(L	-Aver	-H)	(--L	--Aver	--H-)	N	N1	P200
1.0	350.7	0.0	1	C	18		32		0.01	0.11	0.20	15	48	86	31.0	31.0	31.0	5	50	
4.1	347.6	0.2	1	C	20		32		0.01	0.09	0.25	5	61	89		28.0		11	25	
7.1	344.6	0.4	1	C	20		40		0.01	0.09	0.25	5	62	90		43.0		14	23	
10.0	341.7	0.5	1	C	15		28		0.02	0.09	0.25	5	58	84		32.0		14	20	
13.1	338.6	0.6	1	C	15		23		0.02	0.06	0.25	5	66	80		22.4		13	17	
16.0	335.7	0.7	1	C	15		28		0.01	0.06	0.20	15	73	94		39.0		0	0	
19.1	332.6	0.8	1	C	16		28		0.02	0.09	0.25	5	57	83		30.0		10	11	
22.1	329.6	0.9	1	S					0.10	0.14	0.25	5	35	46	15.5	15.5	15.5	16	17	25.8
25.2	326.5	1.0	1	S					0.21	0.23	0.25	5	9	12		4.0		43	44	31.9
31.1	320.6	1.1	1	C					0.02	0:16	0.20	15	28	78		28.0		5	5	
35.5	316.2	1.3	1	C	15		27		0.02	0.09	0.20	15	51	71		27.0		9	8	
37.1	314.6	1.3	1	C	16		31		0.02	0.09	0.20	15	54	77		32.0		3	3	
43.1	308.6	1.5	1	C	18		29		0.05	0.12	0.25	5	49	71		21.5		10	8	
46.2	305.5	1.6	1	S					0.12	0.16	0.25	5	17	24		9.8		18	14	19.4
49.0	302.7	1.7	1	S	17		25		0.09	0.19	0.25	5	18	37		14.0		16	12	17.1
52.0	299.7	1.8	1	C	14		26		0.05	0.14	0.25	5	33	58		10.0		11	8	14.6
55.1	296.6	1.9	1	S	16		27			0.09			45			16.0		11	8	14.5
58.2	293.5	2.0	1	S						0.14			37			15.5		21	15	23.0
61.0	290.7	2.1	1	S					0.19	0.19	0.19		17			7.3		31	22	28.2
64.1	287.6	2.1	1	S						0.24			9			4.5		32	22	24.7
67.0	284.7	2.2	1	S						0.28			10			4.5		37	25	29.3
70.1	281.6	2.3	1	S						0.37			7			3.0		35	23	24.9
73.1	278.6	2.4	1	S						0.34			10			3.6		49	31	32.8
76.0	275.7	2.5	1	S						0.45			6			2.8		24	15	15.9
79.1	272.6	2.6	1	S						0.65			12			4.1		35	22	26.1
85.1	266.6	2.8	1	C	15		33			0.01			78			36.5		15	9	
88.2	263.5	2.9	1	C	15		30			0.01			79			33.5		11	6	
91.0	260.7	3.0	1	C	14		27			0.02			74			28.0		19	11	
97.2	254.5	3.2	1	S						0.28			19			6.7		54	30	32.5
103.0	248.7	3.3	1	S						0.19			28			11.0		35	19	27.3

REQ-05
Barkley dam

Depth	Elev	EUS	Soil		(--%---		)		(------D50------		)	(--P200--		)	(-----P005----		)	(SPT)		(-Nlc--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver---	H--)	(L	-Aver-	H)	(--L	--Aver--	H-)	N	N1	P200
1.0	342.5	0.0	1	C	17		42		0.01	0.09	0.25	5	65	95		47.0		11	>99	
4.1	339.5	0.2	1	C	15		50		0.01	0.09	0.25	5	64	93		48.0		10	26	
7.0	336.5	0.2	1	C	17		40			0.01			94			36.0		8	16	
10.2	333.4	0.3	1	C	16		33		0.01	0.07	0.20	15	63	84		28.0		7	12	
13.1	330.5	0.4	1	C	15		27		0.01	0.08	0.20	15	65	91		26.5		0	0	
16.0	327.5	0.5	1	C	15		26		0.02	0.07	0.20	15	74	96		24.0		0	0	
19.1	324.5	0.6	1	C	16		29		0.01	0.09	0.20	15	54	80		30.0		0	0	
22.2	321.4	0.7	1	C	15		24		0.02	0.08	0.20	15	60	82		25.0		6	7	
25.1	318.5	0.8	1	C	15		25		0.02	0.09	0.20	15	53	75		25.0		5	6	
34.1	309.5	1.1	1	C	18		35		0.03	0.10	0.20	15	50	73		22.0		4	4	

SCS-06

Barkley dam

Depth feet	Elev feet	CUS tsf	Soil #	ID	(--%---) PL	Wn	LL	Wn/LL	(------D50------) L	Aver	H	(--P200--) L	Aver	H	(-----POOS----) L	Aver	H	(SPT) N	N1	(-N1c--) P200
4.0	346.3	0.2	1	C	17		28		0.01	0.11	0.25	5	54	86		27.0		11	26	
7.0	343.3	0.4	1	C	14		30		0.01	0.06	0.20	15	67	87		35.0		6	10	
10.1	340.2	0.4	1	S					0.14	0.18	0.25	5	20	27		11.0		19	28	32.5
13.2	337.1	0.6	1	C	10		26		0.01	0.06	0.20	15	70	91		32.0		7	9	
16.1	334.2	0.6	1	C	16		28		0.01	0.08	0.20	15	61	88		31.0		5	6	
19.1	331.2	0.7	1	C	15		25		0.03	0.09	0.20	15	52	71		23.0		8	9	
25.1	325.2	0.9	1	C	18		28		0.02	0.07	0.20	15	65	87		24.0		2	2	
28.0	322.3	1.0	1	C	17		26		0.03	0.08	0.20	15	57	73		19.0		5	5	11.2
31.1	319.2	1.1	1	C	21		30		0.02	0.09	0.20	15	58	87	27.5	27.5	27.5	0	0	
34.1	316.2	1.2	1	S			15		0.19	0.22	0.25	5	17	28		9.0		24	22	28.7
40.1	310.2	1.4	1	S			17		0.14	0.19	0.25	5	18	26		9.0		20	17	22.8
43.1	307.2	1.5	1	S			20		0.08	0.15	0.20	15	29	44		16.0		7	6	11.3
49.0	301.3	1.6	1	S	20		21		0.07	0.12	0.20	15	38	50		16.5		7	5	11.7
52.1	298.2	1.7	1	S			19			0.21			9			6.3		39	30	32.0
55.1	295.2	1.8	0							0.19			10			5.5		30	22	
58.2	292.1	1.9	1	S			17			0.23			15			7.5		24	17	22.5
61.1	289.2	2.0	1	S			21		0.22	0.22	0.22		22			9.0		20	14	20.0
64.0	286.3	2.1	1	S			18			0.24			17			8.0		22	15	20.5
67.1	283.2	2.2	1	S			16			0.25			15			7.7		20	13	18.0
73.0	277.3	2.4	1	S						0.13			27			10.0		25	16	23.4
76.1	274.2	2.5	1	S	16		27			0.14			37			13.6		23	15	22.5
79.2	271.1	2.6	1	S						0.11			48			18.8		40	25	31.4
85.2	265.1	2.8	1	S						0.22			22			8.5		39	23	30.1
88.0	262.3	3.2	0	_														13	7	
91.1	259.2	3.0	1	C	14		28			0.02		74	74	74		29.4		10	6	
94.1	256.2	3.0	1	C	14		31			0.02			80			32.0		6	3	
97.0	253.3	3.1	1	C	15		29			0.02			72			27.0		8	5	
100.1	250.2	3.2	1	C	16		30			0.02			72			29.0		1	1	
112.2	238.1	3.2	0	_														74	41	

BSB-07

Barkley dam

Depth	Elev	CUS	Soil		(--%---			)	(------	D50	------)	(--	P200	--)	(-----	P005	----)	(SPT)		(-Mic--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	340.5	0.1	3	SC	16	10	26	0.38	0.02	0.70	1.40	13	37	76	4.8	13.9	29.3	21	84	32.5
2.5	339.0	0.1	1	C	18	15	34	0.44	0.01	0.01	0.01		83			35.5		48	99	
4.0	337.5	0.2	1	C	22	20	41	0.49		0.01			94			42.4		29	67	
5.5	336.0	0.3	1	C	23	23	48	0.47		0.01			95			48.5		32	61	
7.0	334.5	0.4	1	C	25	25	49	0.52	0.01	0.05	0.25	5	77	96		49.0		21	34	
8.5	333.0	0.5	1	C	22	24	46	0.52		0.01			95			47.5		15	22	
10.0	331.5	0.6	1	C	22	24	46	0.52	0.00	0.09	0.25	5	67	97		52.0		22	29	
11.5	330.0	0.7	1	C	22	22	45	0.49		0.00			98			51.0		34	42	
13.0	328.5	0.7	1	C	22	23	44	0.51		0.01			90			42.5		26	30	
14.5	327.0	0.8	1	C	20	20	39	0.51		0.00			97			51.0		27	30	
16.0	325.5	0.9	1	C	21	23	40	0.57	0.01	0.04	0.25	5	83	95		45.0		20	21	
17.5	324.0	1.1	2	CS	18	28	36	0.77										14	14	
19.0	322.5	1.2	2	C	18	23	33	0.68	0.01	0.01	0.01	82	83	84	34.0	35.6	37.0	14	13	
20.5	321.0	1.3	2	C	16	23	29	0.80	0.01	0.02	0.02	68	71	80	25.0	27.1	33.0	8	7	
22.0	319.5	1.3	1	C	14	20	25	0.81	0.04	0.09	0.20	15	45	61		23.0		9	8	
23.5	318.0	2.6	2	C	16	23	28	0.82		0.02			78			29.0		7	4	
25.0	316.5	1.5	2	SC	14	21	26	0.82	0.07	0.08	0.08	47	49	51	19.0	20.1	21.0	9	7	
28.0	313.5	1.7	2	CS		23			0.02	[illegible]	[illegible]	41	60	67	17.2	23.5	26.0	9	7	
29.5	312.0	1.7	1	C	15	22	21	1.04	0.08	[illegible]	0.20	15	45	50		15.5		4	3	9.4
31.0	310.5	1.8	1	S		21			0.12	.12	0.12		42			16.0		3	2	8.7
32.5	309.0	1.8	2	CS	17	23	28	[illegible]	0.02	0.11	0.20	15	49	77	13.8	20.9	26.0	8	6	
35.5	306.0	2.6	2	S		19				0.16			29			18.0		11	7	12.5
37.0	304.5	1.9	1	C		18			0.02	0.09	0.20	15	45	65		22.0		9	6	
40.0	301.5	2.6	2	S		24				0.14			18			5.2		18	11	15.7
41.5	300.0	2.1	1	C	15	23	21	1.11	0.06	0.10	0.25	5	44	53		21.0		11	8	
43.0	298.5	2.1	2	S		17			0.12	0.44	1.00	9	28	39	2.0	6.4	8.9	17	12	18.0
44.5	297.0	2.2	1			17				4.10			7					26	18	18.9
46.0	295.5	2.2	1			19				4.27			5			2.0		36	24	24.4
47.5	294.0	2.6	2	S		18				5.80			4			2.9		26	16	16.2
49.0	292.5	2.3	1	S		23				0.23			12			5.7		35	23	27.8
50.5	291.0	2.4	1	S		20				0.26			7					74	48	
52.0	289.5	2.4	1	S		22				0.23			9			4.3		40	25	30.1
53.5	288.0	2.6	0	_														31	19	
55.0	286.5	2.5	1	S		16				0.20			12					37	23	28.0
56.5	285.0	2.6	2	S		21			0.20	0.34	0.44	8	9	10	2.9	3.9	5.3	51	32	31.9
58.0	283.5	2.6	1	S		20				0.31			7			2.9		47	29	30.9
59.5	282.0	2.6	1	S		21				0.60			9		3.8	3.8	3.8	62	39	31.0

BEQ-08

Barkley dam

Depth	Elev	EVS		Soil	(--%---)				(------D50	------)		(--P200--)			(-----P005	----)		(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	348.8	0.0	1	C	17	11	33	0.34	0.02	0.11	0.25	5	51	82		29.5		27	>99	
3.5	346.3	0.2	1	S	16	19	29	0.66	0.13	0.20	0.25	5	21	45		18.0		19	42	32.5
6.0	343.8	0.3	1	S	16	19	28	0.67	0.09	0.18	0.25	5	22	48		18.0		11	20	27.3
8.5	341.3	0.5	1	C	18	21	35	0.60	0.01	0.07	0.25	5	62	83		37.0		11	16	
13.5	336.3	0.8	2	C	16	18	28	0.64	0.02	0.07	0.25	5	54	67	23.7	26.4	27.7	10	11	
16.0	333.8	0.9	1	C	19	19	33	0.58	0.01	0.06	0.25	5	72	88		32.0		17	18	
18.5	331.3	1.0	1	C	18	22	27	0.81	0.01	0.08	0.20	15	64	88		26.0		8	8	
21.0	328.8	1.1	1	C	25	19	45	0.42	0.01	0.09	0.25	5	64	93		47.0		32	31	
23.5	326.3	1.2	1	C	23	33	43	0.77	0.01	0.07	0.25	5	69	93		43.0		23	21	
26.0	323.8	1.2	1	C	20	22	39	0.57	0.01	0.04	0.25	5	74	85		34.0		18	16	
28.5	321.3	1.3	1	C	17	23	30	0.77		0.04		58	58	58	23.0	23.0	23.0	13	11	
31.0	318.8	1.4	1	C	16	25	32	0.77	0.02	0.05	0.25	5	58	67		24.5		10	8	
33.5	316.3	1.5	1	C	17	26	29	0.89	0.03	0.12	0.25	5	39	61		24.8		11	9	
36.0	313.8	1.5	1	C	18	28	34	0.81	0.01	0.09	0.20	15	61	91		38.0		7	6	
38.5	311.3	1.6	2	C	20	26	39	0.67	0.01	0.01	0.01		72			27.0		4	3	
41.0	308.8	1.7	1	C	19	30	38	0.80		0.01			82			27.0		4	3	
43.5	306.3	1.8	2	CS		33			0.02	0.08	0.25	5	58	68		22.0		11	8	
46.0	303.8	1.9	1	S		19			0.25	0.26	0.32	5	5	7				55	40	
48.5	301.3	1.9	1	S					0.17	0.19	0.25	5	12	15				24	17	21.3
51.0	298.8	2.0	1	S		21				0.19			9					34	24	27.6
53.5	296.3	2.1	1	S						0.20			14					24	17	21.4
56.0	293.8	2.2	2	CS	18	23	32	0.71	0.03	0.16	0.29	8	41	74		23.0		13	9	
58.5	291.3	2.2	1	C	21	31	38	0.82		0.02			82			24.5		8	5	
61.0	288.8	2.5	2	C	18	31	36	0.87		0.04			78			20.0		24	15	
63.5	286.3	2.4	1	S						0.40			7					42	27	29.9
66.0	283.8	2.5	1	S		24			0.25	0.25	0.25	7	7	7				43	27	30.0

BCE-09

Barkley dam

Depth	Elev	EVS	Soil	(--%---)				(------D50------)			(--P200--)			(-----P005----)			(SPT)		(-Nlc--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	349.5	0.0	1	C	19	17	38	0.43	0.01	0.07	0.20	15	61	84		31.5		9	90	
3.5	347.0	0.2	2	C	19	18	28	0.65	0.01	0.02	0.02	77	85	90	19.0	19.6	20.0	21	49	
6.0	344.5	0.3	1	C	18	21	31	0.67	0.01	0.06	0.25	5	75	93		27.0		15	27	
8.5	342.0	0.5	2	C	19	23	32	0.73	0.01	0.01	0.01	91	91	91		27.0		17	24	
11.0	339.5	0.7	1	C	16	24	28	0.86		0.02			76			26.0		5	6	
13.5	337.0	0.8	1	C	17	23	30	0.77		0.01			95			33.0		8	9	
16.0	334.5	0.9	1	C	17	23	28	0.81		0.01			86			21.0		7	7	
18.5	332.0	1.0	1	C	17	24	26	0.91	0.02	0.13	0.20	15	44	67		24.0		5	5	
21.0	329.5	1.1	1	C	16	23	30	0.77	0.01	0.01	0.01		73			27.0		5	5	
23.5	327.0	1.1	1	C	17	25	24	1.03	0.04	0.14	0.25	5	38	68		21.0		11	10	
26.0	324.5	1.2	1	C	17	24	24	1.01	0.03	0.06	0.20	15	64	77		19.5		5	5	10.7
28.5	322.0	1.3	1	C	15	20	21	0.97	0.04	0.08	0.25	5	68	84		20.0		14	12	
31.0	319.5	1.4	1	S					0.13	0.20	0.25	5	13	26				24	20	25.4
33.5	317.0	1.5	1	S		21			0.11	0.15	0.25	5	33	43		16.5		18	15	22.8
36.0	314.5	1.5	1	S		23			0.13	0.15	0.25	5	27	30		11.9		31	25	31.1
38.5	312.0	1.6	1	S		23			0.19	0.21	0.25	5	11	14				30	24	28.0
41.0	309.5	1.7	2	S		22			0.08	0.20	0.25	5	16	47		16.5		16	12	16.0
43.5	307.0	1.8	1	S	17	26	31	0.85	0.08	0.10	0.20	15	42	48		14.0		7	5	11.5
46.0	304.5	1.8	1	C	16	26	25	1.02		0.04			56			18.7		12	9	15.4
48.5	302.0	1.9	1	C	16	22	27	0.81		0.03			70			22.0		10	7	
51.0	299.5	2.0	2	C	20	31	42	0.73	0.01	0.04	0.20	15	66	85	27.0	36.7	48.0	6	4	
53.5	297.0	2.1	2	S		23			0.15	0.15	0.16	18	25	31	7.5	11.1	14.0	22	15	21.9
56.0	294.5	2.2	2	C	18	29	36	0.80	0.01	0.02	0.02	71	75	79	25.0	31.1	38.0	10	7	
61.0	289.5	2.3	2	C	17	29	33	0.86	0.02	0.02	0.02	66	72	78	22.0	24.8	28.0	18	12	
63.5	287.0	2.4	1	S		18				0.29			10					52	34	32.5
66.0	284.5	2.5	1	C	18	29	35	0.84		0.03			66			25.5		19	12	
68.5	282.0	2.5	2	S	16	24	30	0.81	0.01	0.13	0.18	16	24	44		16.5		31	19	27.0
73.5	277.0	2.7	1	S		25				0.19			12					51	31	32.5

BEC-10

Barkley dam

Depth feet	Elev feet	EVS tsf	Soil #	ID	PL	Wn	LL	Wn/LL	D50 L	D50 Aver	D50 H	P200 L	P200 Aver	P200 H	P005 L	P005 Aver	P005 H	SPT N	N1	N1c P200
1.0	345.2	0.0	2	C	17	21	32	0.66	0.01	0.08	0.25	5	62	96	24.5	34.0	42.0	24	>99	
2.5	344.7	0.1	1	C	17	22	40	0.54	0.00	0.05	0.25	5	79	99		53.0		17	56	
4.0	343.2	0.2	3	C	20	27	40	0.68	0.00	0.00	0.00	98	99	99	52.0	54.3	58.0	19	39	
5.5	341.7	2.2	3	C	19	22	37	0.60	0.01	0.01	0.01	93	95	97	32.0	37.5	43.0	20	13	
7.0	340.2	0.4	1	C	18	29	34	0.86	0.01	0.15	0.25	5	42	97		33.0		11	18	
8.5	338.7	0.5	2	C		23			0.01	0.18	0.25	5	32	95		36.0		11	16	
10.0	337.2	0.6	3	C	17	25	33	0.76	0.01	0.07	0.20	15	79	95	23.0	28.9	36.5	9	11	
11.5	335.7	0.7	1	C	17	22	30	0.75	0.01	0.16	0.25	5	38	97		32.5		15	19	
13.0	334.2	0.8	2	C	18	24	31	0.77	0.02	0.02	0.02	81	82	83		21.0		8	9	
14.5	332.7	0.8	3	CH	17	24	25	0.97		0.02		62	66	78		21.0		4	4	
16.0	331.2	0.8	1	C					0.02	0.14	0.20	15	38	83		23.0		5	5	
17.5	329.7	0.9	2	S		24			0.09	0.13	0.20	15	34	45		13.0		9	9	16.0
19.0	328.2	1.0	2	CS		25			0.03	0.10	0.20	15	40	61		20.5		8	8	
20.5	326.7	2.2	2	CS	16	24	23	1.05		0.04			60			18.5		5	3	9.6
22.0	325.2	1.0	2	C	18	26	31	0.83	0.02	0.02	0.02	84	86	88	22.0	22.5	23.0	5	5	
23.5	323.7	1.1	2	C	18	27	33	0.82	0.01	0.02	0.02	87	88	90	25.0	27.9	30.5	6	6	
25.0	322.2	1.1	2	C	18	26	34	0.76	0.02	0.05	0.20	15	70	87	18.0	21.5	25.0	8	7	
26.5	320.7	1.2	3	CS	18	26	27	0.96	0.02	0.06	0.17	14	60	83	19.5	21.5	23.5	13	12	
28.0	319.2	1.2	2	CS		23			0.03	0.07	0.14	27	54	67		18.5		10	9	15.5
29.5	317.7	1.3	1	S					0.15	0.17	0.25	5	18	21				39	35	32.5
31.0	316.2	1.3	1	S						0.16			5					31	27	27.1
32.5	314.7	1.4	2	C	21	32	35	0.93	0.03	0.03	0.04	70	72	74	19.5	20.1	21.0	10	8	
34.0	313.2	1.4	2	SC						0.15			18					17	14	19.5
35.5	311.7	1.5	2	SC		25			0.05	0.08	0.09	46	50	59	15.0	15.7	16.0	13	11	17.7
37.0	310.2	1.5	2	C		28			0.02	0.04	0.05	61	67	77	19.0	20.6	23.0	11	9	
38.5	308.7	1.6	2	C	18	28	32	0.87	0.01	0.02	0.04	73	79	84	15.5	28.0	39.0	11	9	
40.0	307.2	1.6	2	C	18	29	33	0.88	0.02	0.03	0.04	66	72	78	24.0	24.3	24.5	11	9	
41.5	305.7	1.6	2	C	16	28	31	0.93	0.03	0.03	0.03	68	68	68	24.0	25.3	26.5	6	5	
43.0	304.2	1.7	1	C		30				0.02			76			30.0		7	5	
44.5	302.7	1.7	2	CS	18	28	31	0.89	0.06	0.28	0.90	2	31	56	0.5	11.0	15.0	18	14	
46.0	301.2	1.8	2	C	18	28	33	0.83	0.02	0.07	0.11	45	59	76	15.5	21.1	27.5	8	6	
47.5	299.7	1.8	2	C	17	27	30	0.91	0.02	0.02	0.02	72	73	75	23.0	25.3	28.0	6	4	
49.0	298.2	1.9	2	C	19	28	36	0.79	0.01	0.01	0.01	76	77	77		28.5		10	7	
52.0	295.2	2.0	2	C	20	31	40	0.77	0.01	0.05	0.09	76	87	96	29.5	31.8	34.5	6	4	
53.5	293.7	2.0	2	C	17	33	30	1.09	0.01	0.55	1.00	26	47	71	13.0	19.9	28.0	14	10	16.5
55.0	292.2	2.1	1	S		20				0.30			9					55	38	31.3
56.5	290.7	2.1	1	S		21				0.23			6					27	19	19.6
58.0	289.2	2.1	1	S		26			0.22	0.22	0.22		5			4.5		25	17	17.2
59.5	287.7	2.2	1	S		18				0.26			10			4.0		83	56	32.5

BCD-11

Barkley dam

Depth	Elev	EVS	Soil		(--%---			)	(------D50	------	)	(--P200	--	)	(----PC05	----	)	(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver---	H--)	(L	-Aver-	H)	(--L	--Aver--	H-)	N	N1	P200
1.0	346.0	0.0	2	C	18	25	37	0.67	0.01	0.10	0.20	15	56	94	37.5	39.3	41.0	5	32	
3.5	343.5	0.2	2	C	18	25	39	0.65	0.01	0.10	0.20	15	56	92	28.0	31.8	35.5	4	9	
6.0	341.0	0.4	3	C	18	23	34	0.69	0.00	0.01	0.01	91	94	97	34.5	44.3	51.5	14	23	
8.5	338.5	0.5	3	C	17	22	32	0.71	0.01	0.05	0.20	15	71	89	20.0	29.6	33.0	8	11	
11.0	336.0	0.7	3	C	16	23	24	0.93	0.01	0.03	0.05	68	83	90	16.8	22.7	25.5	5	6	
13.5	333.5	0.8	3	C	15	20	24	0.83	0.03	0.04	0.05	61	69	79	15.0	17.3	18.5	5	6	
16.0	331.0	0.9	2	C	17	21	21	1.00	0.06	0.12	0.20	15	38	56		18.0		9	10	16.4
18.5	328.5	0.9	3	C	16	23	30	0.77	0.01	0.01	0.01	87	88	91	29.0	31.3	33.8	7	7	
21.0	326.0	1.0	3	C	16	22	26	0.87	0.01	0.03	0.05	65	76	88	19.8	23.2	29.0	8	8	
23.5	323.5	1.1	3	C	17	24	26	0.93	0.03	0.05	0.06	56	68	78	16.0	16.2	16.5	8	8	14.0
26.0	321.0	1.2	3	C	17	24	26	0.93	0.03	0.03	0.04	70	75	84	19.0	19.8	21.0	4	4	9.9
28.5	318.5	1.2	3	C	19	27	35	0.77	0.02	0.05	0.20	15	69	89	18.0	22.0	23.5	3	3	
31.0	316.0	1.3	3	CS	21	31	38	0.82	0.01	0.03	0.09	38	79	97	11.0	24.4	29.0	3	3	
33.5	313.5	1.4	3	C	17	28	33	0.86	0.01	0.03	0.05	58	69	86	21.0	23.6	27.4	8	7	
36.0	311.0	1.5	2	CS	13	28	17	1.67	0.02	0.14	0.25	5	43	91	8.4	19.1	28.0	13	11	17.7
38.5	308.5	1.5	2	CS	19	30	26	1.16	0.02	0.11	0.25	5	48	77		28.8		14	11	
41.0	306.0	1.6	2	CS	19	30	36	0.82	0.02	0.07	0.25	5	64	83		30.5		10	8	
43.5	303.5	1.7	3	C	18	30	31	0.94	0.02	0.03	0.06	53	66	77	18.0	28.3	21.5	0	0	
46.0	301.0	1.8	2	S		23			0.20	0.23	0.25	5	6	8				29	22	22.9
48.5	298.5	1.9	2	S		23				0.20			25			13.5		19	14	20.4
51.0	296.0	2.2	2	S		23				0.19			10					15	10	12.1
53.5	293.5	2.2	2	S		19				4.40			11					27	18	21.5
56.0	291.0	2.2	2	S		21				0.24			10					31	21	23.9
58.5	288.5	2.2	2	S		14			0.20	0.22	0.25	9	20	27		9.5		35	24	30.1

BEQ-12

Barkley dam

Depth	Elev	EVS	Soil		(--%---			)	(------D50	------	)	(--P200	--	)	(------P005	----	)	(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	347.5	0.0	2	CS	18	21	41	0.52	0.01	0.07	0.20	15	53	72		34.0		8	41	
3.5	345.0	0.2	2	C	20	23	42	0.55	0.00	0.06	0.25	5	75	94	37.0	43.8	50.5	15	34	
6.0	342.5	0.3	2	C	19	22	44	0.50	0.01	0.06	0.25	5	77	95	42.0	43.3	44.5	20	34	
8.5	340.0	0.5	3	C	17	23	32	0.74	0.01	0.01	0.01	93	95	97	33.0	36.8	40.5	14	19	
11.0	337.5	0.6	3	C	15	21	25	0.84	0.01	0.04	0.09	86	90	93	28.5	32.3	38.5	11	14	
13.5	335.0	0.7	3	C	17	21	31	0.67	0.01	0.01	0.01	91	92	93	34.0	34.8	35.5	14	17	
16.0	332.5	0.8	2	C	17	25	28	0.88	0.01	0.09	0.20	15	57	88	29.0	29.3	31.0	6	7	
18.5	330.0	0.8	3	CS	16	21	24	0.86	0.02	0.06	0.10	42	56	83	18.5	19.4	24.0	9	10	
21.0	327.5	0.9	2	S		16			0.22	0.23	0.25	5	9	17	3.3	4.0	4.9	39	41	31.9
23.5	325.0	1.0	1	S		20			0.24	0.25	0.25	5	7	9		4.5		22	22	23.9
26.0	322.5	1.1	3	SM		20			0.07	0.12	0.20	15	38	53	11.3	13.7	17.2	8	8	14.2
28.5	320.0	1.1	2	S		21			0.08	0.18	0.25	5	22	48		12.1		27	25	31.2
31.0	317.5	1.2	2	S		19			0.21	0.22	0.25	5	8	10	3.8	4.1	4.4	24	22	24.1
33.5	315.0	1.3	2	S		18			0.18	0.20	0.25	5	14	19		6.5		27	24	29.2
36.0	312.5	1.4	2	S		21			0.13	0.16	0.25	5	25	37	9.3	9.7	10.0	20	17	24.3
38.5	310.0	1.4	2	S	16	20	21	0.93	0.02	0.18	0.25	5	32	85	7.0	17.5	28.0	16	13	20.7
41.0	307.5	1.5	3	C	19	23	28	0.82	0.03	0.05	0.06	52	62	76	17.0	20.4	22.3	7	6	
43.5	305.0	1.6	3	C	19	23	32	0.72	0.05	0.10	0.25	5	50	65		18.8		14	11	
46.0	302.5	1.7	2	S		23			0.16	0.20	0.25	5	12	21		5.6		35	27	31.8
48.5	300.0	1.8	3	SC	18	25	26	0.94	0.04	0.08	0.12	20	46	65	11.8	16.7	21.0	12	9	15.6
51.0	297.5	1.8	3	S	55	22	95	0.23	0.06	0.16	0.28	14	36	54	6.3	12.9	18.5	19	14	
53.5	295.0	2.1	3	MC	18	22	28	0.78	0.06	0.07	0.07	51	52	57	15.0	16.7	21.0	17	12	18.9
56.0	292.5	2.0	2	S		20			0.20	0.20	0.21	10	10	11		5.3		31	22	25.7
58.5	290.0	2.1	2	S		22			0.12	0.16	0.19	12	23	36		13.0		33	23	30.1
61.0	287.5	2.1	2	S		17			0.22	0.22	0.23	6	14	22	3.6	6.8	10.0	23	16	20.3

BCS-13

Barkley dam

Depth	Elev	EVS	Soil		(--%---			-)	(------	D50	------)	(--	P200	--)	(-----	P005	----)	(SPT)		(-Nlc--)
feet	feet	tsf	#	ID	Fl	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	343.0	0.0	2	C	18	21	43	0.49	0.01	0.15	0.25	5	40	92		42.4		13	74	
2.5	341.5	0.1	2	C	20	23	43	0.54	0.01	0.05	0.20	15	74	90	40.0	41.7	43.3	8	22	
4.0	340.0	0.2	2	C	18	22	36	0.62	0.01	0.07	0.20	15	64	91	40.5	40.8	41.0	9	19	
5.5	338.5	0.3	3	CS	15	22	26	0.83	0.01	0.05	0.09	43	66	82	13.2	21.8	31.0	12	22	
7.0	337.0	0.3	3	C	20	25	42	0.59	0.03	0.09	0.20	15	49	70	24.0	25.9	27.4	7	12	
8.5	335.5	0.4	2	S	12	23	24	0.94	0.10	0.19	0.25	5	24	46	8.8	14.9	21.0	11	18	25.1
10.0	334.0	0.4	2	C	16	23	28	0.83	0.01	0.05	0.20	15	67	82	30.0	31.8	34.0	3	5	
11.5	332.5	0.5	2	C	17	23	28	0.83	0.02	0.11	0.20	15	51	87		26.0		4	6	
13.0	331.0	0.5	2	C	17	26	26	1.01	0.02	0.07	0.20	15	60	84	18.0	23.7	30.5	3	4	
14.5	329.5	0.6	3	CS	16	26	23	1.14	0.02	0.07	0.12	44	62	76	16.8	19.7	22.0	5	7	12.9
16.0	328.0	0.6	2	S		19			0.11	0.17	0.25	5	28	42		13.0		12	15	22.7
17.5	326.5	0.6	2	S					0.20	0.23	0.25	5	12	23		7.6		21	26	30.9
19.0	325.0	0.7	2	S		18			0.19	0.22	0.25	5	12	19	5.2	6.3	7.5	21	25	30.2
20.5	323.5	0.7	2	S		20			0.19	0.23	0.25	5	8	16		3.5		22	26	29.3
22.0	322.0	0.8	2	S		16			0.09	0.13	0.20	15	37	47	12.5	14.3	15.8	7	8	14.3
23.5	320.5	0.8	2	S		20			0.19	0.22	0.25	5	11	18		7.0		24	26	30.6
25.0	319.0	0.9	2	S		21			0.20	0.22	0.25	4	8	15	2.5	3.7	5.0	39	42	19.8
26.5	317.5	0.9	2	CS	17	19	32	0.59	0.04	0.16	0.25	5	25	61		23.0		23	24	
28.0	316.0	1.0	3	C	16	22	29	0.74	0.01	0.01	0.01	78	80	82	31.5	32.0	32.5	6	6	
29.5	314.5	1.0	3	C	17	22	29	0.75	0.01	0.02	0.03	71	74	76	23.0	25.0	27.0	[illegible]	6	
31.0	313.0	1.1	2	C	16	21	29	0.75	0.02	0.06	0.20	15	60	75	24.0	25.5	27.0	7	7	
34.0	310.0	1.1	1	C		23			0.02	0.15	0.20	15	30	70		27.0		7	7	
35.5	308.5	1.2	1	C	16	22	29	0.77	0.04	0.15	0.20	15	35	76		24.0		8	7	
37.0	307.0	1.2	2	C	18	25	30	0.83	0.03	0.11	0.20	15	47	76	20.0	20.8	21.5	5	4	
38.5	305.5	1.3	3	CM	17	21	28	0.76	0.05	0.08	0.25	5	54	72	17.5	19.9	22.0	12	11	
40.0	304.0	1.3	2	MS		23			0.05	0.12	0.20	15	35	55	12.7	15.0	16.5	9	8	14.2
41.5	302.5	1.4	3	S	18	24	30	0.82	0.14	0.20	0.25	5	20	40	9.0	12.6	15.5	13	11	16.0
43.0	301.0	1.4	3	MC	16	22	24	0.93	0.07	0.12	0.20	15	37	52	11.8	16.7	23.5	8	7	13.0
44.5	299.5	1.5	3	C	19	24	27	0.88	0.03	0.08	0.20	15	53	68	14.5	17.6	23.5	7	6	
46.0	298.0	1.5	3	CS	18	22	31	0.72	0.02	0.03	0.03	63	71	77	20.0	21.5	22.5	8	6	
47.5	296.5	1.6	2	SM	19	25	24	1.03	0.05	0.07	0.09	50	52	54	17.0	17.5	18.0	5	4	10.2
49.0	295.0	1.6	3	SC	16	22	27	0.86	0.05	0.09	0.16	23	43	54	10.6	16.2	20.0	10	9	14.2
50.5	293.5	1.9	3	SC	19	23	43	0.53	0.01	0.10	0.18	13	46	89		37.4		28	20	
52.0	292.0	1.7	2	S		20			0.19	0.20	0.21	10	10	10		4.4		49	37	32.5
53.5	290.5	1.8	2	S		19			0.18	0.18	0.19	14	23	31	8.3	8.6	9.0	36	27	32.4
55.0	289.0	1.8	2	S		21			0.18	0.21	0.24	15	18	24		9.0		24	18	23.8
56.5	287.5	1.8	2	S		18			0.19	0.20	0.21	17	18	19	3.9	7.6	11.3	45	33	32.5
58.0	286.0	1.9	2	S		14			0.24	1.72	3.20	6	9	12	3.7	4.4	5.1	45	32	32.3
59.5	284.5	1.9	2	S		17			0.20	0.31	0.41	5	7	9		1.6		25	18	19.4

ECO-14
Barkley dam

Depth	Elev	CUS	Soil		(--%---			)	(------D50	------)	(	--P200--	)(	-----P005	----)	(	SPT)	(	-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver---	H--)	(L	-Aver-	H)	(--L	--Aver--	H-)	N	N1	P200
3.5	341.5	0.2	3	C	18	16	30	0.53	0.01	0.05	0.25	5	75	92	28.0	29.4	31.0	21	46	
6.0	339.0	0.3	2	C	20	20	33	0.60	0.01	0.07	0.25	5	69	92	30.0	30.8	31.5	11	19	
8.5	336.5	0.4	2	C	19	21	32	0.65	0.02	0.13	0.25	5	42	66	18.0	19.8	22.0	15	24	
11.0	334.0	0.5	3	CS	16	21	31	0.67	0.01	0.05	0.15	26	59	78	12.5	21.4	27.0	19	27	
13.5	331.5	0.6	2	C	17	23	30	0.77	0.01	0.10	0.20	15	55	86		30.5		8	11	
16.0	329.0	0.6	3	C	18	27	37	0.74	0.01	0.01	0.01	94	95	96	34.0	36.8	39.0	5	6	
18.5	326.5	0.7	3	C	18	26	34	0.76	0.01	0.01	0.01	89	94	98	31.0	34.5	36.5	6	7	
21.0	324.0	0.8	3	SC		19			0.03	0.13	0.17	31	42	67	13.0	14.9	21.0	11	12	19.7
23.5	321.5	2.1	0	__														10	7	
26.0	319.0	1.0	3	C	18	25	28	0.91	0.02	0.06	0.20	15	65	84	22.0	22.5	23.0	6	6	
28.5	316.5	1.0	2	C	17	26	27	0.96	0.02	0.07	0.20	15	64	84	26.0	28.3	31.0	3	3	
31.0	314.0	1.1	3	S		20			0.09	0.20	0.25	5	14	43		15.5		15	14	18.6
33.5	311.5	2.1	2	S		25				0.14			24			11.0		10	7	12.0
38.5	306.5	1.3	2	S		20			0.14	0.23	0.29	5	23	35		10.0		11	10	14.6
41.0	304.0	1.4	2	C	17	23	37	0.64	0.01	0.06	0.25	5	68	84		42.0		10	8	
43.5	301.5	2.1	0	__														9	6	
46.0	299.0	1.6	3	C	19	23	27	0.86	0.02	0.06	0.20	15	63	76	15.0	17.0	18.0	4	3	
48.5	296.5	1.6	2	S		22			0.08	0.10	0.11	43	46	48	14.5	15.6	17.0	7	5	11.7
51.0	294.0	1.7	3	CS	20	30	31	0.95	0.03	0.03	0.03	62	62	63	23.0	24.1	25.5	11	8	
53.5	291.5	2.1	3	CS		33				0.02			82			29.0		1	1	
56.0	289.0	2.1	3	SC		32				0.09		45	45	45		13.0		17	12	19.1
58.5	286.5	1.9	2	CS	17	20	30	0.68	0.04	0.11	0.22	20	39	53	7.3	15.9	22.0	22	16	
61.0	284.0	2.0	2	S		24			0.19	0.23	0.27	10	11	13		5.0		26	18	21.9
63.5	281.5	2.1	1							2.50			13			3.6		22	15	19.0

ECO-15

Sarkley dam

Depth	Elev	EVS	Soil		<--%---				<------D50------>			<--P200-->			<-----P005---->			(SPT)			<-Mlc-->
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	<--L	--Aver---	H-->	<L	-Aver-	H>	<--L	--Aver--	H->	N	N1	i	P200
19.5	347.2	1.1	2	C		23			0.00	0.16	0.25	5	39	96		53.0		13	12		
21.0	344.7	1.3	3	S		21			0.00	0.14	0.25	5	45	98	25.0	45.1	52.0	21	19		
23.5	342.2	1.4	3	C		16			0.01	0.08	0.25	5	57	84	28.0	31.5	35.0	42	35		
26.0	339.7	1.6	2	C		17			0.01	0.09	0.25	5	56	91	30.0	33.0	36.0	35	28		
28.5	337.2	1.7	2	C		18			0.02	4.30	7.50	4	19	68	4.0	10.7	27.0	89	69		32.5
31.0	334.7	1.8	3	C		24			0.00	0.07	0.25	5	71	96	49.0	50.0	51.0	30	23		
33.5	332.2	1.8	2	C		24			0.01	0.09	0.25	5	66	97	43.0	44.0	45.0	28	21		
36.0	329.7	1.9	3	C		23			0.00	0.04	0.25	5	84	97	39.0	44.7	52.0	28	20		
38.5	327.2	2.0	3	C		21			0.01	0.01	0.01	95	97	98	34.0	35.4	37.0	29	21		
41.0	324.7	2.1	3	C		22			0.01	0.07	0.25	5	66	93	28.0	32.5	34.0	28	19		
43.5	322.2	2.1	3	C	19	23	32	0.74	0.01	0.02	0.02	62	73	89	24.5	26.0	27.0	12	8		
46.0	319.7	2.2	2	S	17	23	22	1.05	0.11	0.16	0.25	5	27	39	15.0	15.8	16.5	11	7		12.9
48.5	317.2	2.3	2	CS	17	25	27	0.94	0.03	0.13	0.20	15	37	67	10.0	17.1	21.0	6	4		10.2
51.0	314.7	2.4	3	CS	15	23	21	1.09	0.04	0.07	0.11	42	51	59	16.5	18.6	20.0	9	6		12.1
53.5	312.2	3.4	4	C	16	23	26	0.87	0.02	0.02	0.03	66	71	75	22.0	25.0	28.0	11	6		
56.0	309.7	2.5	3	CS	16	24	25	0.96	0.02	0.06	0.20	15	58	76	14.5	23.0	26.0	7	4		
58.5	307.2	2.6	3	C	17	24	24	0.98	0.01	0.03	0.04	65	78	89	19.5	25.6	30.0	8	5		
61.0	304.7	2.7	2	SC		23			0.04	0.18	0.25	5	25	61		19.0		14	9		13.8
63.5	302.2	3.4	5	CS		25			0.02	0.08	0.17	27	52	70	20.0	24.3	26.5	17	9		
66.0	299.7	2.8	2	SC		23			0.03	0.22	0.25	5	15	60		23.0		40	24		
68.5	297.2	2.9	2	S		26			0.34	1.17	2.00	4	6	7				48	28		29.5
71.0	294.7	3.0	1	S		26				0.17			12					25	15		17.8
73.5	292.2	3.0	2	S		23			0.18	0.27	0.38	10	11	11				43	25		29.3
76.0	289.7	3.1	2	CS		26			0.02	0.09	0.18	21	57	85		32.0		33	19		
78.5	287.2	3.2	2	S	15	24	27	0.88	0.11	0.16	0.22	6	26	45		16.5		44	25		30.8
81.0	284.7	3.3	3	S		20			0.28	0.48	2.00	4	9	12		7.5		61	34		32.3
83.5	282.2	3.4	3	SC		22			0.04	0.49	1.05	6	25	59	12.0	13.9	21.5	22	12		18.0
86.0	279.7	3.4	2	S		24			0.27	0.27	0.28	6	6	7				42	23		23.9

CCO-16

Barkley dam

Depth	Elev	EVS	Soil		(--%---				(------D50------			(--P200--			(-----P005----			(SPT)		(-Nic--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver---	H--)	(L	-Aver	-H)	(--L	--Aver--	H-)	N	N1	P200
34.0	331.7	3.0	0	_														18	10	
40.0	325.7	1.7	2	C		22			0.01	0.09	0.25	5	63	94	37.0	37.8	38.5	23	18	
41.5	324.2	1.8	2	C		26			0.01	0.09	0.25	5	54	80	29.0	31.0	33.0	17	13	
43.0	322.7	1.8	3	C	14	24	25	0.94	0.02	0.04	0.20	15	62	76	24.5	26.3	27.5	9	7	
44.5	321.2	1.9	3	C	14	23	25	0.90	0.01	0.03	0.07	51	72	89	22.0	29.4	40.0	12	9	
46.0	319.7	1.9	2	CS		23			0.05	0.16	0.25	5	29	59	13.7	18.6	22.5	13	9	15.4
47.5	318.2	1.9	3	CS	15	23	26	0.90	0.03	0.10	0.20	15	45	68	14.5	19.2	23.5	6	4	10.5
49.0	316.7	2.0	3	C	15	24	27	0.86	0.03	0.07	0.20	15	51	62	17.5	18.9	20.5	3	2	
50.5	315.2	2.0	3	S		21			0.09	0.14	0.20	15	33	48	13.0	14.7	16.5	6	4	10.3
52.0	313.7	2.1	3	C	15	22	26	0.85	0.03	0.06	0.20	15	56	66	19.5	21.5	25.0	2	1	
53.5	312.2	2.1	3	CS	16	23	24	0.96	0.06	0.13	0.20	15	40	54	15.1	16.5	17.5	4	3	9.1
55.0	310.7	2.2	3	C	16	23	24	0.94	0.04	0.07	0.20	15	52	64	19.5	21.0	22.5	6	4	
56.5	309.2	3.0	4	CS	16	24	29	0.83	0.04	0.07	0.09	47	56	78	20.1	20.2	20.5	6	3	
58.0	307.7	2.3	3	C		25			0.01	0.04	0.20	15	74	88	29.3	30.7	32.0	0	0	
59.5	306.2	3.0	4	CS	16	24	28	0.85	0.01	0.07	0.19	26	61	82	9.0	20.7	28.0	11	6	
61.0	304.7	2.4	3	SC	15	26	27	0.98	0.03	0.15	0.20	15	33	61	7.8	13.1	20.0	9	6	11.9
62.5	303.2	2.4	3	C	14	24	22	1.09	0.03	0.07	0.25	5	49	57	21.5	22.0	22.5	12	8	
64.0	301.7	2.5	3	SC	14	22	24	0.93	0.03	0.19	0.25	5	20	61	3.3	11.7	24.5	34	22	29.3
65.5	300.2	2.5	2	S		22			0.14	0.23	0.26	5	12	39	2.0	8.0	14.0	35	22	26.7
67.0	298.7	2.5	3	S		16			0.25	3.29	5.50	6	7	11	1.2	1.6	3.5	88	55	
68.5	297.2	2.6	2	S		18			0.21	2.11	4.00	6	9	11	1.9	2.3	2.8	74	46	29.3
70.0	295.7	2.6	3	S		26			0.14	0.23	0.38	1	15	23	4.0	4.1	4.2	31	19	24.4
71.5	294.2	2.7	2	CS	19	24	35	0.70	0.03	0.12	0.28	19	53	73	6.5	18.4	25.5	27	17	
73.0	292.7	2.7	2	S		22			0.18	0.30	0.39	7	8	10	2.8	3.3	4.0	48	29	31.2
74.5	291.2	2.8	3	CS	18	25	34	0.72	0.02	0.15	0.28	10	60	82	5.5	18.3	24.5	21	13	
76.0	289.7	2.8	2	CS	16	28	30	0.92	0.03	0.05	0.10	46	61	71	17.0	22.9	26.5	23	14	
77.5	288.2	2.9	3	CS	17	25	34	0.75	0.02	0.05	0.14	29	62	79	9.5	20.1	25.5	19	11	
79.0	286.7	2.9	3	CS	17	25	31	0.82	0.02	0.05	0.17	21	64	75	8.5	22.5	25.5	21	12	
80.5	285.2	3.0	2	S		19			0.28	0.29	0.31	6	7	8	2.0	3.2	4.0	59	34	27.9
82.0	283.7	3.0	1	S		20				1.80			4			1.5		34	20	19.8
83.5	282.2	3.0	0	_														28	16	

EC3-17

Barkley Dam

Depth	Elev	EVS	Soil*		(--%---)				(------D50	------	------)	(--P200	--	--)	(-----P005	----	----)	(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
33.5	332.6	1.8	2	C		25			0.00	0.09	0.25	5	59	96	41.0	46.5	52.0	28	21	
36.0	330.1	1.9	2	C		24			0.01	0.09	0.25	5	64	93	42.0	42.3	42.5	11	8	
38.5	327.6	2.0	3	C		21			0.01	0.01	0.01	70	75	81	31.5	34.0	35.5	23	16	
41.0	325.1	2.1	3	C		24			0.01	0.01	0.01	84	86	89	36.0	36.6	37.5	19	13	
43.5	322.6	2.2	3	C		24			0.00	0.01	0.01	81	85	92	34.5	47.2	68.0	11	7	
46.0	320.1	2.2	3	C		26			0.01	0.01	0.01	78	88	94	34.0	36.3	38.0	9	6	
48.5	317.6	2.3	3	C	16	26	27	0.96	0.01	0.02	0.03	59	77	90	25.5	34.0	40.0	6	4	
51.0	315.1	3.4	4	C		26			0.01	0.01	0.02	77	79	81	30.5	31.9	33.0	5	3	
53.5	312.6	2.5	3	CS	17	22	28	0.80	0.02	0.09	0.25	5	48	69	8.5	19.9	25.5	15	10	
56.0	310.1	3.4	4	C	17	24	29	0.84	0.01	0.01	0.02	64	80	88	19.5	28.6	32.5	9	5	
58.5	307.6	3.4	4	C	18	30	30	0.99	0.01	0.02	0.06	51	80	93	14.5	26.2	32.0	8	4	
61.0	305.1	2.7	3	SC	23	25	37	0.69	0.02	0.19	0.25	5	18	69		20.5		57	35	
63.5	302.6	2.8	2	S		21			0.25	0.32	0.41	5	7	7				55	33	29.2
66.0	300.1	2.8	1	S		22			0.14	0.21	0.25	5	11	21				28	17	20.2
68.5	297.6	2.9	2	CS		24			0.01	0.09	0.19	16	55	87		29.5		20	12	
71.0	295.1	3.0	2	S		21			0.23	0.27	0.30	13	13	14	6.5	6.9	7.2	39	23	27.7
73.5	292.6	3.1	3	CS		29			0.01	0.03	0.18	13	79	93	25.5	33.0	42.0	16	9	
76.0	290.1	3.1	3	CS	18	26	32	0.82	0.01	0.09	0.20	15	73	87	23.5	29.0	32.0	10	6	
78.5	287.6	3.2	2	S		18			0.19	0.24	0.27	7	15	26		12.5		42	23	29.2
81.0	285.1	3.3	3	S		17			0.25	0.39	0.65	6	7	7				98	54	
83.5	282.6	3.4	2	S		18			0.29	1.02	1.50	6	6	7				72	39	
86.0	280.1	3.4	2	S		21			0.28	0.29	0.31	6	8	10				50	27	30.3

ECS-15

Barkley dam

Depth	Elev	CUS	Soil	(--%---			)	(------D50	------	------)	(--P200	--	)	(-----P005	----	)	(SPT)		(-N1c--)
feet	feet	tsf	% ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
26.0	340.1	1.6	3 SC		21			0.01	0.23	0.31	5	23	92		45.0		32	25	
28.5	337.6	1.7	2		20			0.25	7.48	13.00	1	3	5	0.1	1.5	3.0	39	29	29.5
31.0	335.1	1.8	2 C		21			0.01	0.10	0.25	5	59	94	38.0	43.6	48.0	38	29	
33.5	332.6	1.8	2 C		24			0.00	0.09	0.25	5	63	95	47.0	51.0	55.0	30	22	
36.0	330.1	1.9	3 C		20			0.01	0.04	0.25	5	75	89	39.0	42.2	48.0	22	16	
39.5	327.6	2.0	3 C		22			0.01	0.01	0.01	68	75	82	32.0	34.8	38.0	20	14	
41.0	325.1	2.1	3 C		23			0.01	0.01	0.01	75	82	90	28.0	30.8	34.0	17	12	
43.5	322.6	2.2	3 C	14	22	27	0.82	0.01	0.03	0.04	55	70	87	22.3	26.7	33.8	12	6	
43.5	322.6	2.1	3 C	14	22	27	0.80	0.02	0.09	0.25	5	50	78	22.3	26.1	30.0	12	8	
46.0	320.1	3.4	4 C		25			0.01	0.01	0.02	64	82	94	25.3	33.3	36.5	9	5	
48.5	317.6	2.3	3 C		25			0.01	0.02	0.02	71	78	88	27.0	29.5	33.0	8	5	
51.0	315.1	2.4	3 C		21			0.02	0.02	0.03	62	73	86	23.5	27.2	32.0	6	4	
53.5	312.6	3.4	5 SC		18			0.02	0.13	0.19	31	47	71	14.8	20.2	29.0	15	8	
56.0	310.1	2.5	3 C		20			0.01	0.04	0.20	15	80	94	32.0	35.1	38.0	8	5	
58.5	307.6	2.6	3 CS		24			0.01	0.12	0.20	15	45	84	27.5	29.7	31.0	8	5	
61.0	305.1	2.7	2 SC		24			0.02	0.17	0.25	5	31	71		27.0		36	22	
63.5	302.6	2.8	2 S		22			0.25	0.30	0.34	5	5	6				49	30	30.0
66.0	300.1	2.8	2 S		23			0.11	0.20	0.25	5	16	47		22.5		25	15	
68.5	297.6	2.9	2 SC		24			0.01	0.12	0.20	10	44	87		34.5		24	14	
71.0	295.1	3.0	2 S		20			0.21	0.24	0.30	10	11	11				39	23	26.5
73.5	292.6	3.1	2 CS		23			0.01	0.08	0.19	7	58	93		37.0		18	10	
76.0	290.1	3.4	4 CS		20			0.02	0.06	0.18	13	62	83	7.0	20.5	26.5	10	5	
78.5	287.6	3.2	3 S		22			0.21	0.24	0.27	6	11	33		15.0		49	27	31.1
81.0	285.1	3.3	2 S		17			4.50	4.73	5.00	5	5	6				73	40	
83.5	282.6	3.4	2 S		19			0.36	1.28	2.20	5	6	7				66	36	3.9

BCS-19

Barkley dam

Depth	Elev	EVS	Soil		(--%---				(------D50------			(--P200--			(-----P005----			(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
41.0	323.4	1.9	3	SC		18			0.01	1.00	2.20	5	21	94		35.5		18	13	
43.5	320.9	2.0	2	C		22			0.01	0.09	0.25	5	64	94	40.0	41.0	42.0	17	12	
46.0	318.4	2.1	3	C		22			0.01	0.06	0.25	5	72	91	34.0	38.0	44.0	26	18	
48.5	315.9	3.2	4	C		26			0.01	0.01	0.01	83	85	88	33.0	36.3	39.0	9	5	
51.0	313.4	2.3	3	C		26			0.01	0.01	0.01	85	87	91	27.5	30.3	32.0	8	5	
53.5	310.9	2.3	3	C	19	26	35	0.75	0.01	0.01	0.02	86	89	93	22.0	25.9	32.0	7	5	
56.0	308.4	2.4	3	C	21	29	40	0.74	0.01	0.05	0.25	5	74	92	24.0	26.3	28.0	12	8	
58.5	305.9	2.5	2	S		26			0.19	0.21	0.25	5	11	15		6.0		34	22	25.4
61.0	303.4	2.6	2	S		22			0.15	0.20	0.25	5	18	33	9.0	9.9	11.0	26	16	22.0
63.5	300.9	2.6	2	SC	17	27	31	0.86	0.02	0.13	0.25	5	30	69	4.0	11.0	21.0	18	11	17.6
66.0	298.4	2.7	2	S		25			0.19	0.19	0.19	9	11	14		3.0		21	13	15.6
68.5	295.9	2.8	2	S		31			0.12	0.14	0.16	21	28	35	4.0	6.5	9.0	15	9	14.7
71.0	293.4	2.9	2	SC	18	25	32	0.77	0.03	0.17	0.25	10	37	81		16.5		15	9	15.4
73.5	290.9	3.0	3	C	17	26	31	0.84	0.02	0.03	0.03	72	75	78	15.5	16.8	18.5	8	5	
76.0	288.4	3.0	3	SC	16	23	33	0.68	0.03	0.26	0.31	6	15	55	4.5	7.5	16.5	34	20	25.2
78.5	285.9	3.1	2	S		19			0.30	1.62	2.50	5	6	6				47	27	28.3
81.0	283.4	3.2	2	S		20			0.26	3.30	4.60	5	6	8				44	25	26.3

BSD-22

Barkley Dam

Depth feet	Elev feet	EOS tsf	Soil #	Soil ID	(--%---) PL	Wn	LL	Wn/LL	(------D50------) L	Aver	H	(--P200--) L	Aver	H	(-----P005----) L	Aver	H	(SPT) N	N1	(-N1c--) P200
41.5	323.2	2.0	3	C		21			0.01	0.04	0.25	5	80	95	32.0	32.2	33.0	21	15	
43.0	321.7	2.1	3	C		21			0.09	0.16	0.25	5	55	95		36.0		18	12	
44.5	320.2	3.2	3	C		21				0.01		93	94	95	38.0	40.3	44.0	22	12	
52.0	312.7	3.2	4	C		24			0.01	0.01	0.02	65	89	94	23.0	32.0	34.0	9	5	
56.5	308.2	3.2	3	C	21	30	44	0.69		0.01			92			23.5		9	5	
58.0	306.7	2.5	2	S		25			0.18	0.21	0.25	5	7	9				30	19	20.1
59.5	305.2	2.6	3			22			0.20	1.21	5.00	5	14	56		10.0		23	14	18.5
61.0	303.7	2.6	2	S		24			0.17	0.21	0.25	5	10	15				24	15	17.2
62.5	302.2	3.2	5	CS	18	27	31	0.88	0.02	0.06	0.11	29	52	70	21.5	21.8	23.0	9	5	
64.0	300.7	2.7	2	SC		28			0.14	0.20	0.25	5	9	13				19	12	13.2
65.5	299.2	2.8	2	SC		25			0.19	0.22	0.25	5	8	11				30	18	20.0
67.0	297.7	2.8	3	SC		25			0.01	0.14	0.19	9	28	72		30.0		28	17	
68.5	296.2	2.9	2	SC		26				0.16		14	14	14				25	15	19.1
70.0	294.7	2.9	2	SC		24				0.23			8					40	23	26.0
71.5	293.2	3.0	2	C	19	24	33	0.72		0.02			81			22.0		30	17	
74.5	290.2	3.2	4	CS		26			0.16	0.17	0.21	28	66	70		30.0		13	7	
76.0	288.7	3.1	3	SC		22			0.08	0.23	0.31	14	26	50		14.0		20	11	17.2
77.5	287.2	3.1	1	S		19				3.00			5					37	21	21.3
79.0	285.7	3.2	2	S		21				0.40			7					42	23	24.9

BCD-21

Barkley dam

Depth feet	Elev feet	CVS tsf	#	Soil ID	(--%---) PL	Wn	LL	Wn/LL	(------D50------) L	Aver	H	(--P200--) L	Aver	H	(-----PC05----) L	Aver	H	(SPT) N	N1	(-N1c--) P200
1.0	340.5	0.0	1	C						0.25		5	28	75				16	>99	32.5
3.5	338.0	0.2	2	C						0.25		5	65	95				27	63	32.5
6.0	335.5	0.3	2	C						0.25		5	66	97				13	22	30.7
8.5	333.0	0.5	3	C		27			0.01	0.01	0.01	92	95	97	37.0	42.5	48.0	8	11	
11.0	330.5	0.7	3	C									97					14	17	25.4
13.5	328.0	0.8	3	C									95					15	16	24.7
16.0	325.5	1.0	3	C						0.25		5	79	90				12	12	19.4
18.5	323.0	1.1	3	C		22			0.01	0.01	0.01	78	81	84	28.0	28.5	29.0	7	7	
21.0	320.5	1.3	3	C	16	21	28	0.74	0.01	0.06	0.20	15	67	83	23.0	25.3	29.0	6	5	
23.5	318.0	1.4	3	C	15	23	26	0.90	0.02	0.06	0.20	15	59	70	23.5	23.8	24.0	3	3	
26.0	315.5	1.5	2	C	16	23	26	0.90	0.02	0.08	0.20	15	51	77	22.0	23.6	25.5	4	3	
28.5	313.0	1.5	3	C	17	22	29	0.74	0.02	0.05	0.20	15	67	82	20.5	22.4	24.0	6	5	
31.0	310.5	1.6	1	C	15	26	22	1.17	0.04	0.15	0.20	15	28	55		17.7		4	3	8.7
33.5	308.0	1.7	3	CS	16	24	29	0.82	0.02	0.13	0.20	15	37	72	6.0	17.0	21.5	6	5	
36.0	305.5	1.7	1	C		28			0.02	0.16	0.20	15	25	65		19.0		7	5	10.4
38.5	303.0	1.8	2	C	16	28	25	1.11	0.02	0.08	0.20	15	54	71		19.0		5	4	9.9
41.0	300.5	1.9	2	CS	16	23	24	0.96	0.03	0.13	0.20	15	33	61	8.0	14.3	18.0	9	6	12.7
43.5	298.0	2.0	2	S		19			0.17	3.12	5.00	6	12	21		10.5		33	23	28.0
46.0	295.5	2.1	2	S		18			3.00	3.70	4.40	4	5	5				55	38	
48.5	293.0	2.1	1	S		25				0.20			7					22	15	16.0
51.0	290.5	2.2	2	S		23			0.25	0.26	0.26	7	8	8				33	22	24.1
53.5	288.0	2.3	2	SC		26			0.03	0.32	0.44	5	21	63		22.0		11	7	
56.0	285.5	2.4	2	S		21			0.20	0.21	0.22	10	11	12				30	19	23.0
58.5	283.0	2.5	2	S		20			0.27	1.57	2.35	5	5	5				34	22	22.2
61.0	280.5	2.5	2	S		20			0.40	1.07	2.20	7	14	18		6.0		32	20	25.4

BSD-22

Bartley dam

Depth	Elev	EVS	Soil*		(--%---)				(------D50------)			(--P200--)			(-----P005----)			(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
22.0	319.5	1.3	3	CS	15	20	23	0.89	0.05	0.11	0.20	15	59	97		19.5		8	7	
23.5	318.0	1.4	3	C	15	23	27	0.85		0.02		53	57	66		21.5		3	3	
25.0	316.5	2.4	4	S	16	23	23	1.00	0.09	0.10	0.12	38	43	45	14.5	15.4	16.5	5	3	9.5
26.5	315.0	1.5	3	CS		26			0.01	0.10	0.20	15	49	83	13.5	22.6	27.5	3	2	
28.0	313.5	1.5	2	CS		25			0.02	0.13	0.20	15	36	73		25.0		8	6	
29.5	312.0	1.6	2		15	23	24	0.97	0.05	0.14	0.20	15	31	53	10.5	15.0	19.5	6	5	10.7
31.0	310.5	1.6	3	S		22			0.09	0.12	0.20	15	39	47	12.5	16.2	17.5	4	3	9.4
32.5	309.0	1.7	3	C	15	23	25	0.93	0.02	0.06	0.20	15	56	71	21.5	23.8	28.5	1	1	
34.0	307.5	1.7	3	SC		23			0.03	0.16	0.20	15	26	61	17.0	18.7	21.5	5	4	9.0
35.5	306.0	1.8	3	CS	15	23	27	0.86	0.03	0.14	0.26	15	34	64	19.2	19.8	20.0	9	7	
37.0	304.5	2.4	4	CS	16	25	28	0.89	0.02	0.05	0.14	35	59	69	18.0	20.7	23.0	8	5	
38.5	303.0	2.4	4	CS		26						13	56	77				6	4	10.1
40.0	301.5	2.4	4			24				0.13		4	53	85				10	6	12.7
41.5	300.0	2.4	5	SC		23			0.04	0.11	0.18	13	37	61	5.5	13.8	20.0	10	6	12.7
43.0	298.5	2.0	3	SM		22			0.03	0.24	0.29	11	18	62		22.0		19	13	
46.0	295.5	2.4	2	S		15							10					45	29	32.0
47.5	294.0	2.1	2	S		19				4.00		4	4	5				38	26	26.2
49.0	292.5	2.2	2	S		24			0.21	0.23	0.24	8	9	10				32	22	24.4
50.5	291.0	2.2	2	S		24				0.12		6	7	7				24	16	17.1
52.0	289.5	2.2	1	S		24				0.10			7					23	15	16.5
53.5	288.0	2.3	2	SC		21				0.05		4	22	57		20.0		18	12	
55.0	286.5	2.3	2	S		24			0.10	0.19	0.24	7	22	47		16.0		23	15	21.2
56.5	285.0	2.4	2			19				5.40			6					24	16	15.9
58.0	283.5	2.4	2	S		20			0.30	2.79	6.10	6	7	8				33	21	22.6

950-23

Barkley dam

Depth	Elev	EUS		Soil	<--%	---	---	--->	(------	D50	------)	(--	P200	--)	(-----	POCS	----)	(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver	---H--)	(L	-Aver	-H)	(--L	--Aver	--H-)	N	N1	P200
18.5	323.8	1.1	3	CS	24	23	48	0.48	0.00	0.09	0.25	5	61	95	53.0	54.7	55.0	20	19	
26.0	321.3	1.3	1	C	18	23	33	0.70	0.01	0.15	0.20	15	32	80		35.0		3	3	
28.5	318.8	1.4	1	C	17	24	30	0.78	0.01	0.15	0.20	15	32	77		33.0		7	6	
31.0	316.3	1.5	1	C	20	27	38	0.71	0.01	0.14	0.20	15	41	93		37.0		9	7	
33.5	313.8	1.6	3	C	18	25	31	0.81	0.01	0.02	0.04	55	74	82	24.0	31.1	35.0	7	6	
36.0	311.3	1.6	3	C	17	24	31	0.76	0.01	0.02	0.03	68	80	87	23.5	30.0	33.0	5	4	
38.5	308.8	1.7	2	C	17	23	32	0.71	0.02	0.08	0.20	15	53	78	26.5	26.5	26.6	9	7	
41.0	306.3	2.3	0	_														5	3	
43.5	303.8	1.9	3	C	23	33	44	0.77	0.01	0.02	0.03	73	79	85	24.0	28.2	31.0	8	6	
46.0	301.3	2.0	3	C	18	36	33	1.08	0.02	0.03	0.04	54	60	68	19.5	23.1	25.0	8	6	
48.5	298.8	2.0	3	CS	20	33	37	0.91	0.01	0.09	0.19	21	51	79	8.0	20.9	33.0	14	10	
51.0	296.3	2.1	3	SC		26			0.01	0.13	0.30	9	47	88	3.0	20.2	38.0	23	16	
53.5	293.8	2.2	2	S		19			0.70	1.38	1.95	4	4	5	1.0	1.5	2.0	28	19	19.1
56.0	291.3	2.3	3	CS	21	28	40	0.69	0.01	0.67	2.00	11	56	85	14.9	26.2	33.0	16	11	
58.5	288.8	2.3	3	C	19	29	36	0.82	0.03	0.09	0.17	50	65	83	22.0	23.3	24.0	12	8	

[illegible]

Barkley Dam

Depth feet	Elev feet	EVS tsf	Soil #	ID	(--Z---) PL	Wn	LL	Wn/LL	(------D50 L	Aver	H------)	(--P200 L	Aver	H--)	(-----P005 L	Aver	H----)	(SPT) N	N1	(-N1c--) P200
3.5	343.8	0.2	2	C		19				0.25			5					17	39	
6.0	341.3	0.3	1	C		20				0.25			5					12	22	21.7
26.5	320.8	1.4	3	C	20	21	42	0.51	0.01	0.01	0.01	91	93	96	40.0	41.1	43.0	22	19	
29.0	318.3	1.4	3	C	18	24	38	0.63	0.01	0.01	0.01	90	90	91	36.0	37.4	38.0	16	13	
31.5	315.8	1.5	3	C	19	26	39	0.67		0.01			93			38.0		12	10	
41.5	305.8	1.8	3	C	16	24	31	0.76	0.02	0.02	0.03	62	69	76	24.0	26.0	28.0	8	6	
64.0	283.3	2.6	2	SC						0.29			11					86	54	32.5
66.5	280.8	2.6	1	S						1.50			8					30	19	20.4

BCQ-25

Barkley dam

Depth feet	Elev feet	EVS tsf	Soil #	Soil ID	(--%---) PL	Wn	LL	Wn/LL	(------D50------) L	Aver	H	(--P200--) L	Aver	H	(-----P005-----) L	Aver	H	(SPT) N	N1	(-N1c--) P200
2.5	347.3	2.4	0	_														0	0	
11.0	338.8	2.4	0	_														13	8	
13.5	336.3	0.7	2	C	15	18	29	0.62	0.02	0.13	0.25	5	48	70		26.5		14	17	
16.0	333.8	0.8	2	C		16				0.25		5	48	90				19	21	30.5
18.5	331.3	0.9	3	C		22				0.20		15	71	90				9	10	16.3
21.0	328.8	0.9	2	C		20				0.25		5	53	95				25	26	31.7
23.5	326.3	1.0	2	C		22				0.25		5	50	95				23	23	30.9
26.0	323.8	1.1	3	C		24							85					15	14	22.1
28.5	321.3	1.2	3	SC		23			0.11	0.12	0.16	30	42	47	21.0	33.1	38.0	11	10	
31.0	318.8	1.3	3	SC	17	25	27	0.93	0.03	0.11	0.20	15	37	54	16.0	20.3	21.5	6	5	
33.5	316.3	1.3	3	C	17	28	30	0.92	0.01	0.02	0.02	72	75	76	24.0	24.5	25.5	5	4	
36.0	313.8	1.4	1	C		29			0.02	0.15	0.20	15	30	73		24.5		5	4	
38.5	311.3	1.5	2	C	18	28	38	0.73	0.01	0.15	0.20	15	47	95		35.5		8	7	
41.0	308.8	1.6	2	C	17	27	32	0.87	0.02	0.08	0.20	15	54	78	20.5	22.0	23.5	4	3	
43.5	306.3	1.6	2	S		27			0.01	0.18	0.25	5	16	44		14.0		13	10	14.3
46.0	303.8	1.7	2		16	21	25	0.83	0.06	0.87	2.30	5	21	54		14.0		13	10	14.8
48.5	301.3	1.8	3	CS		33			0.01	0.08	0.25	5	61	93	34.0	35.1	36.0	13	10	
51.0	298.8	2.4	3	SC	16	27	30	0.91	0.03	0.08	0.16	8	41	65		21.0		13	8	
53.5	296.3	1.9	2	S		20			0.28	0.28	0.29	10	11	12				23	17	19.8
56.0	293.8	2.0	3	SC		24			0.03	0.15	0.24	10	27	62		17.5		20	14	20.9
58.5	291.3	2.1	3	C	19	30	33	0.91	0.01	0.02	0.02	72	83	94	22.0	26.0	30.0	6	4	
61.0	288.8	2.2	3	CS	18	30	31	0.97	0.01	0.04	0.14	21	67	89	11.0	24.0	37.0	13	9	
63.5	286.3	2.3	3	SC		23			0.01	0.26	0.34	4	21	54	14.0	21.1	30.0	23	15	
66.0	283.8	2.3	2	S		21				0.22			13					28	18	23.0
68.5	281.3	2.4	2	S		19			0.19	1.69	2.89	5	7	9				30	19	20.6

BCG-26

Barkley dam

Depth	Elev	EUS	Soil		(--%---			)	(------D50		------)	(--P200		--)	(-----P005		----)	(SPT	)	(-Nlc--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver	---H--)	(L	-Aver	-H)	(--L	--Aver	--H-)	N	N1	P200
14.5	335.3	0.7	1	C		17			0.02	0.17	0.25	5	25	66		28.0		13	15	
16.0	333.8	0.8	3	C		18				0.25		5	61	97				13	15	22.5
17.5	332.3	0.8	2	C		20				0.25		5	59	97				10	11	18.0
19.0	330.8	0.9	2	C	20	22	30	0.74	0.01	0.11	0.20	15	65	95		28.0		6	6	
20.5	329.3	0.9	2	C		20				0.25		5	62	98				24	25	31.4
22.0	327.8	1.0	2	C		22				0.25		5	61	93				24	24	31.2
23.5	326.3	1.0	2	C		24				0.25		5	65	95				15	15	22.9
25.0	324.8	1.1	3	C		24				0.25		5	79	91				16	15	23.6
26.5	323.3	1.1	3	C		23				0.25		5	68	86				16	15	23.2
28.0	321.8	1.2	2	CS	16	21	26	0.82	0.16	0.19	0.25	5	43	65		23.0		13	12	
29.5	320.3	2.4	4	CS		22			0.03	0.06	0.15	29	59	67		26.0		8	5	
31.0	318.8	1.3	3	SC	15	23	27	0.86	0.04	0.11	0.15	33	40	54	24.0	27.7	29.5	9	8	
32.5	317.3	1.3	3	C		26				0.01		74	86	95		29.0		6	5	
34.0	315.8	1.4	3	CS		27				0.12		29	83	87				6	5	11.4
35.5	314.3	2.4	4	CS	18	28	30	0.94		0.03		13	67	98		25.6		7	5	
37.0	312.8	1.4	3	C	37	26			0.01	0.01	0.01	83	91	96	28.2	31.2	35.0	3	2	
38.5	311.3	1.5	2	C		27			0.02	0.10	0.20	15	51	79		24.0		7	6	
40.0	309.8	2.4	4	CS	18	25	31	0.82	0.02	0.03	0.08	48	74	80	24.0	28.1	31.0	4	3	
41.5	308.3	1.6	3	CH	18	27	31	0.86	0.03	0.04	0.06	53	67	85	16.4	19.8	22.8	4	3	
43.0	306.8	2.4	5	CS		32			0.02	0.07	0.13	37	77	90	8.5	15.8	21.2	9	6	12.1
44.5	305.3	1.7	2	S		20			0.22	1.43	6.20	5	8	12				32	25	28.0
46.0	303.8	1.7	2	S		20				0.25		5	10	33				13	10	12.2
47.5	302.3	1.8	3	SC		26			0.01	0.15	0.25	5	30	87	12.0	15.1	34.0	11	8	14.2
49.0	300.8	1.8	2	SC		28			0.17	0.21	0.25	5	31	87		8.9		17	13	19.8
50.5	299.3	1.8	1	S		28			0.17	0.22	0.25	5	8	14		4.5		17	13	14.0
52.0	297.8	2.4	4	CS		29			0.02	0.15	0.25	26	48	73	20.0	23.3	27.5	20	13	
53.5	296.3	1.9	1	S		21							32					14	10	16.5
55.0	294.8	2.0	2	S		24			0.22	0.27	0.35	10	11	11	4.4	6.3	7.5	14	10	12.4
56.5	293.3	2.0	2	C	18	25	29	0.86		0.03		62	64	67		24.0		8	6	
58.0	291.8	2.1	3	C		30						84	88	93				3	2	8.5
59.5	290.3	2.4	3	CS	19	27	31	0.88	0.03	0.05	0.12	37	67	73	12.0	22.2	24.0	7	5	
61.0	288.8	2.2	3	CS		31						17	81	92				9	6	12.4
64.0	285.8	2.3	2	SC		20							10					37	25	29.0
65.5	284.3	2.3	1	S		22				0.35			7					22	15	15.5
67.0	282.8	2.4	2			36			0.00	0.11	0.22	7	47	87		62.0		24	16	

BCQ-27

Barkley dam

Depth	Elev	CUS	Soil		(--%---)				(------D50	------)		(--P200	--)		(-----P005	----)	(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
16.0	326.7	1.0	3	C		21			0.02	0.02	0.03	58	71	95	28.3	29.4	30.5	13	13	
18.5	324.2	1.1	3	C		25			0.01	0.01	0.01	77	86	93	33.6	36.8	40.0	11	10	
21.0	321.7	1.3	3	CS	15	24	26	0.94	0.01	0.04	0.09	50	67	87	22.4	29.0	38.5	5	4	
23.5	319.2	1.4	3	CS	15	24	31	0.77	0.02	0.10	0.20	15	40	59	13.0	24.1	27.6	6	5	
26.0	316.7	2.5	5	CS	14	23	25	0.90	0.02	0.05	0.19	17	51	61	21.0	23.7	26.4	5	3	
28.5	314.2	1.6	2			27			0.04	0.17	0.20	9	21	58		20.2		3	2	
31.0	311.7	1.6	2	C		29			0.01	0.06	0.20	15	68	96	30.0	32.7	35.4	3	2	
33.5	309.2	2.5	4	CS		25			0.04	0.06	0.11	39	52	58	14.2	18.5	20.5	3	2	8.4
36.0	306.7	1.8	2			26			0.02	0.10	0.20	15	44	73	13.0	21.0	28.9	5	4	
38.5	304.2	1.9	1	S		23			0.16	0.22	0.25	5	12	25		7.5		13	10	12.2
41.0	301.7	2.0	3	CS	15	26	24	1.07	0.03	0.10	0.20	15	42	63	22.5	23.6	24.5	6	4	
43.5	299.2	2.0	2			24			0.17	0.21	0.25	5	11	17		5.2		17	12	14.5
46.0	296.7	2.1	2	S	14	19	22	0.87	0.08	2.61	4.42	6	24	50		21.0		58	40	
53.5	289.2	2.5	0	_														22	14	
56.0	286.7	2.5	0	_														27	17	
58.5	284.2	2.5	1	S		22				0.30			5					40	25	25.5

SCO-29

Barkley dam

Depth	Elev	EVS	Soil		(--Z---)				(------D50------)			(--P200--)			(-----P005----)			(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver	---H--)	(L	-Aver	-H)	(--L	--Aver	--H-)	N	N1	P200
16.0	326.7	1.0	3	C		21						63	76	93				12	12	19.4
17.5	325.2	1.1	3	C		23				0.02		65	75	82		25.0		8	8	
19.0	323.7	1.2	3	C		23				0.02		68	84	96		27.0		10	9	
20.5	322.2	1.2	2	C	15	24	25	0.94	0.05	0.12	0.20	15	49	68		20.6		5	4	
22.0	320.7	1.4	3	C	18	24	31	0.79		0.01		79	85	88		34.0		6	5	
23.5	319.2	1.4	1	S		22				0.11			42			16.0		3	3	8.9
25.0	317.7	2.4	5	SC	14	23	26	0.88	0.03	0.09	0.11	40	44	60	13.5	16.5	23.0	5	3	9.5
26.5	316.2	2.4	6	CS		24				0.03		15	56	73		23.0		4	3	
28.0	314.7	1.6	3	SC	15	25	26	0.97	0.03	0.14	0.20	4	41	72		24.5		6	5	
29.5	313.2	1.6	2	C	16	27	31	0.87	0.01	0.08	0.20	15	61	89	24.0	26.0	28.0	1	1	
31.0	311.7	1.7	3	C		26			0.02	0.08	0.20	15	73	87		25.0		3	2	
32.5	310.2	1.7	3	C	15	26	24	1.07	0.03	0.03	0.04	55	58	61	19.9	22.2	24.5	2	2	
34.0	308.7	1.8	3	C		24			0.04	0.15	0.20	15	56	82		20.5		2	2	
35.5	307.2	1.8	2	S		25			0.08	0.14	0.20	15	33	48	12.8	14.8	17.8	5	4	9.9
37.0	305.7	1.8	1	C		22			0.05	0.21	0.25	5	16	59		21.5		10	7	
38.5	304.2	1.9	2	S		23			0.12	0.17	0.20	15	28	42		15.0		5	4	9.1
40.0	302.7	1.9	2	CS	15	25	23	1.07	0.05	0.14	0.20	15	29	56		21.0		7	5	
41.5	301.2	2.0	2	S		23			0.10	0.17	0.20	15	26	45		15.0		5	4	8.8
43.0	299.7	2.0	3	CS		23			0.06	0.21	0.25	5	28	63	20.0	22.7	24.0	10	7	
44.5	298.2	2.1	3	S		23			0.06	0.11	0.15	27	33	37	11.3	13.9	16.0	6	4	10.2
46.0	296.7	2.1	2	C		20						10	38	66				44	30	33.0
47.5	295.2	2.2	2	C		16							8					X132	90	
49.0	293.7	2.2	2	C		17							6					67	45	
53.5	289.2	2.3	2	S		24			0.21	0.25	0.28	6	10	17				23	15	!7.7
55.0	287.7	2.4	1	C	17	29	28	1.02		0.07			52			20.2		11	7	
56.5	286.2	2.4	3	CS		28			0.01	0.05	0.19	12	65	78	17.9	33.0	37.0	13	8	

ECG-29

Barkley dam

Depth feet	Elev feet	EVS tsf	Soil* #	ID	(--%---) PL	Wn	LL	Wn/LL	(------D50------) L	Aver	H	(--P200--) L	Aver	H	(-----P005-----) L	Aver	H	(SPT) N	N1	(-N1c--) P200
23.5	323.7	1.4	3	C		23			0.01	0.05	0.25	5	82	97	39.0	39.5	41.0	14	12	
26.0	321.2	1.6	3	C	13	23	25	0.91	0.02	0.09	0.25	5	52	80	24.8	28.0	32.0	10	8	
28.5	318.7	1.7	3	C	14	23	26	0.89	0.02	0.02	0.03	56	65	71	26.2	28.0	29.9	7	5	
31.0	316.2	2.8	3	C		26				0.02		44	67	67	18.0	26.5	26.5	2	1	
33.5	313.7	1.9	3	C		27			0.01	0.02	0.02	73	78	84	30.0	32.3	34.0	4	3	
36.0	311.2	1.9	2		15	26	23	1.12	0.01	0.09	0.20	15	50	85	21.0	28.0	35.0	4	3	
38.5	308.7	2.8	4	CS	16	26	27	0.96	0.02	0.05	0.17	22	60	75	21.5	22.8	25.0	7	4	
41.0	306.2	2.1	2	S		23			0.16	0.21	0.25	5	15	30				10	7	10.8
43.5	303.7	2.2	2	SC		24			0.02	0.21	0.25	5	13	62		23.0		15	10	
46.0	301.2	2.2	2	S		23			0.23	0.24	0.25	5	9	23				24	16	18.3
48.5	298.7	2.8	6	S		38			0.01	0.10	0.18	21	48	76		35.0		14	9	
51.0	296.2	2.4	2	S		20			0.23	1.98	4.60	5	7	8				28	18	19.4
53.5	293.7	2.5	2	S		20				0.26		8	9	10				37	23	26.6
56.0	291.2	2.6	2	C		32			0.02	0.02	0.02	69	74	78	28.0	28.9	30.0	12	8	
58.5	288.7	2.6	2	SC	16	25	24	1.02	0.04	0.11	0.16	16	31	53		18.0		22	14	20.8
63.5	283.7	2.8	2	S		21			0.41	0.46	0.49	6	6	7				25	15	15.7

REG-30
Barkley dam

Depth	Elev	EVS	Soil		(--Z---)				(-------D50-------		)	(--P200--)			(-----P005----		)	(SPT)		(-Nlc--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver	---H--)	(L	-Aver	-H)	(--L	--Aver	--H-)	N	N1	P200
1.0	346.7	2.8	0	_														0	0	
25.0	322.7	1.5	3	C	14	21	27	0.79		0.04		53	60	72		24.8		7	6	
26.5	321.2	1.6	3	CS	14	21	25	0.85	0.06	0.08	0.11	44	51	56	22.3	24.0	24.8	6	5	
28.0	319.7	1.7	3	C		23						64	67	72				5	4	10.0
29.5	318.2	1.7	2		15	23	26	0.87	0.03	0.12	0.20	15	38	57		23.0		6	5	
31.0	316.7	1.8	3	CS		23			0.05	0.12	0.20	15	43	63		19.5		5	4	10.0
32.5	315.2	1.8	2	C		26				0.20		15	53	72				4	3	9.3
34.0	313.7	1.9	2			26				0.20		15	48	83				3	2	8.6
35.5	312.2	1.9	3	CS		24			0.11	0.14	0.20	15	56	77		17.5		6	4	10.5
37.0	310.7	2.0	3	C	15	24	24	1.01	0.02	0.07	0.20	15	49	63	21.0	22.6	24.5	3	2	
38.5	309.2	2.0	2	C	14	24	22	1.11	0.14	0.17	0.20	15	36	53		17.0		5	4	9.8
40.0	307.7	2.0	1	S		24			0.18	0.19	0.20	15	20	30		12.1		9	6	10.8
41.5	306.2	2.1	2	C	17	25	28	0.90	0.02	0.13	0.20	15	45	72		23.0		6	4	
44.5	303.2	2.2	3	C		25			0.01	0.11	0.25	5	51	83	30.0	33.9	37.0	13	9	
46.0	301.7	2.2	1	S		24			0.25	0.25	0.25	5	6	7				31	21	21.4
47.5	300.2	2.3	3	SC	32	41	58	0.71	0.24	0.25	0.25	5	38	90				10	7	
49.0	298.7	2.4	3	CS		25			0.32	5.41	10.50	3	26	56				49	32	32.5
50.5	297.2	2.4	1			21				6.90			5					37	24	24.2
52.0	295.7	2.4	2	S		17				0.27		12	14	16				48	31	32.5
53.5	294.2	2.5	2	S		25				0.19		14	15	15				21	13	17.7
55.0	292.7	2.5	3	SC		29						31	62	87				12	8	13.9
56.5	291.2	2.6	3	CS	17	30	34	0.88		0.02		69	72	77		23.0		6	4	
58.0	289.7	2.6	1	S		24				0.14		27	27	27				17	11	16.3
59.5	288.2	2.7	2	S		21			0.23	0.24	0.24	7	8	8				49	30	31.5
61.0	286.7	2.7	2	_		15						19	15	20				59	36	32.5
62.5	285.2	2.8	2	S		20				0.60		10	10	10				23	14	16.2

BCD-31
Barkley dam

Depth	Elev	EUS	Soil		(--%---			)	(------D50	------	)	(--P200	--	)	(-----P005	----	)	(SPT)		(-N1c--)
feet	feet	tsf	%	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
8.5	341.5	2.6	0	_														22	14	
16.0	334.0	1.0	3	C	17	21	30	0.69	0.01	0.10	0.25	5	61	89	22.6	32.0	39.0	15	15	
18.5	331.5	1.0	2	C		30				0.25		5	66	97				11	11	17.9
31.0	319.0	1.4	3	C		25			0.01	0.02	0.02	67	77	95	29.0	30.7	32.0	9	8	
33.5	316.5	1.5	3	CS		26			0.01	0.06	0.14	36	67	86	22.0	29.5	35.2	8	7	
36.0	314.0	1.6	3	C		27			0.01	0.05	0.20	15	75	96	36.2	52.4	73.0	5	4	
38.5	311.5	1.6	3	CS	15	24	19	1.29	0.02	0.11	0.20	15	40	66	22.0	23.2	25.5	6	5	
41.0	309.0	1.7	2	C	18	33	35	0.94	0.01	0.07	0.20	15	62	81	25.0	26.5	28.0	3	2	
43.5	306.5	1.8	3		27	40	46	0.87	0.02	0.02	0.02	73	78	83	22.3	24.0	25.0	4	3	
46.0	304.0	1.9	2	C		34			0.00	1.90	5.00	5	40	97		52.0		22	16	
48.5	301.5	1.9	2	CS		27			0.14	0.20	0.25	5	11	18				14	10	12.5
51.0	299.0	2.6	5	CS		32			0.01	0.04	0.12	33	76	97	31.0	37.0	40.0	10	6	
53.5	296.5	2.1	2	S		25				0.21		13	15	17				28	19	24.9
56.0	294.0	2.2	2	S		24			0.14	0.24	0.28	13	22	43		20.0		31	21	
58.5	291.5	2.3	3	CS	18	27	30	0.93	0.02	0.04	0.08	47	66	79	22.0	24.0	26.0	11	7	
61.0	289.0	2.3	3	CS		30			0.01	0.05	0.14	29	70	90	31.4	32.4	35.0	17	11	
63.5	286.5	2.4	2	S		17			2.30	3.10	3.90	5	7	8				27	17	18.4
68.5	281.5	2.6	2	S		22			0.24	2.30	5.40	5	7	8				28	18	18.6

ESB-32
Bartley dam

Depth	Elev	CVS	Scr!		(--%---)				(------D50	------)		(--P200--)			(-----P005	----)		(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver---	H--)	(L	-Aver-	H)	(--L	--Aver--	H-)	N	N1	P200
29.5	320.5	1.4	3	C		24				0.25		5	61	86				15	13	20.4
31.0	319.0	1.4	3	C		26						65	75	80				9	8	13.9
32.5	317.5	1.5	2	CS	15	25	25	1.00	0.15	0.16	0.20	15	49	75		19.0		8	7	12.9
34.0	316.0	1.5	3	C		26				0.25		5	73	85				11	9	15.5
35.5	314.5	2.4	4	CS	18	29	33	0.89	0.01	0.09	0.14	26	71	84		26.2		7	5	
37.0	313.0	1.6	3	C		25			0.01	0.06	0.20	15	72	89		36.7		4	3	
38.5	311.5	1.7	3	C		27			0.01	0.06	0.20	15	64	85	30.8	31.1	31.2	2	2	
40.0	310.0	2.4	3	CS		32						45	51	55				4	3	8.9
41.5	308.5	1.7	2	CS	19	32	37	0.88	0.02	0.12	0.20	15	40	79				5	4	
43.0	307.0	1.8	3	C		37				0.20		15	60	70				5	4	10.0
44.5	305.5	1.8	3	CS	23	33	43	0.77	0.02	0.17	0.20	15	43	80		25.0		8	6	
46.0	304.0	1.9	2			30				0.25		5	35	89				26	19	28.1
47.5	302.5	1.9	2			27			0.14	0.19	0.25	5	31	54		11.5		18	13	20.1
49.0	301.0	2.0	3	CS		34			0.01	0.12	0.20	15	45	93		40.0		7	5	
50.5	299.5	2.0	2	C		32				0.25		5	43	85				10	7	13.4
52.0	298.0	2.1	3	CS		25			0.10	0.18	0.23	16	51	70		42.6		20	14	
53.5	296.5	2.1	3	S		23			0.21	0.25	0.30	10	25	36		16.8		19	13	19.2
55.0	295.0	2.1	2	S		25				0.27		9	9	9				36	25	28.3
56.5	293.5	2.2	2	S		21				0.38		7	17	23				40	27	32.5
58.0	292.0	2.2	3	CS	17	27	29	0.91	0.02	0.06	0.10	32	53	67		26.1		10	7	
59.5	290.5	2.3	3	C		29				0.02		55	63	78		28.0		9	6	
61.0	289.0	2.3	2	C		30						60	65	70				15	10	16.5
62.5	287.5	2.4	2	CS		24				0.48		10	48	56				41	27	31.9
64.0	286.0	2.4	1	_		19							15					38	24	30.1

BEQ-33

Barkley dam

Depth	Elev	EVS	Soil		(--%---			---)	(------	D50	------)	(--	P200	--)	(-----	P005	----)	(SPT)		(-Nlc--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
43.0	319.9	2.0	3	SC		22			0.08	0.14	0.25	5	30	60		23.6		12	8	
44.5	318.4	2.1	3	C		27			0.01	0.11	0.25	5	69	85		35.0		11	6	
46.0	316.9	2.1	2	C		25				0.20		15	67	97				9	6	12.5
47.5	315.4	2.2	3	C		27						90	93	97				7	5	11.0
49.0	313.9	2.2	3	C		26				0.20		15	79	95				5	3	9.6
50.5	312.4	2.3	2	C		25				0.20		15	67	95				6	4	10.2
52.0	310.9	2.3	3	C		27			0.01	0.05	0.20	15	74	90	30.1	39.0	43.0	6	4	
53.5	309.4	3.0	4	CS		26			0.01	0.08	0.15	25	62	81		30.7		9	5	
55.0	307.9	2.4	3	CS	19	29	38	0.76	0.02	0.07	0.20	15	55	80		26.8		7	5	
56.5	306.4	2.4	3	S		29			0.02	0.21	0.25	5	22	79		25.0		21	13	
58.0	304.9	3.0	4	SC	19	24	34	0.70	0.02	0.09	0.16	10	29	79	25.4	25.4	25.4	16	9	
59.5	303.4	2.5	3	MS		29			0.01	0.15	0.25	5	32	80		28.5		14	9	
61.0	301.9	2.6	2	CS		30				0.25		5	30	85				12	7	13.4
62.5	300.4	2.6	2	S		25			0.12	0.19	0.25	5	14	34		14.4		20	12	16.4
64.0	298.9	2.7	2	S		24				0.16		5	11	18				27	17	19.7
65.5	297.4	2.7	2	SC		31			0.01	0.12	0.21	19	53	95	6.5	19.7	36.3	23	14	21.7
67.0	295.9	2.8	3	SC		22			0.02	0.18	0.26	19	36	88	14.2	21.4	30.0	18	11	
68.5	294.4	2.8	2	S		25			0.22	0.23	0.24	12	12	13				24	14	17.9
70.0	292.9	2.9	3	CH		30				0.01		80	92	95		30.0		8	5	
71.5	291.4	2.9	2	C		29			0.01	0.01	0.01	73	81	89	34.2	35.3	36.5	11	6	
73.0	289.9	2.9	2			26			0.02	0.05	0.08	46	60	74	15.0	20.4	25.8	18	10	
74.5	288.4	3.0	3	SC		23				0.23		7	15	25				47	27	32.7

900-34

Sardis Dam

Depth	Elev	EVS	Soil		(--%---)				(------D50------)			(--P200--)			(-----P005----)			(SPT)		(-N1c--)
feet	feet	tsf	#	ID	Pl	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P20C
43.5	319.2	2.0	3	CS	16	23	36	0.65	0.01	0.05	0.20	15	66	79		31.0		9	6	
46.0	316.7	2.1	3	C	19	28	36	0.76	0.01	0.01	0.01	81	85	87	28.5	30.9	33.2	6	4	
48.5	314.2	2.2	3	C	17	26	32	0.82	0.01	0.01	0.02	74	84	92	29.5	32.8	34.5	7	5	
51.0	311.7	2.3	3	C	17	26	30	0.87	0.01	0.02	0.02	72	78	89	27.0	30.6	39.0	6	4	
53.5	309.2	2.3	2		16	26	31	0.83	0.02	0.14	0.25	5	36	71	12.2	17.7	23.2	10	7	12.8
56.0	306.7	3.1	4	C	20	30	37	0.80	0.02	0.04	0.12	41	59	69	12.0	18.0	22.0	12	7	
58.5	304.2	2.5	2	CS	16	24	28	0.86	0.06	0.18	0.25	5	22	52	8.0	13.1	17.0	20	13	
61.0	301.7	2.6	3	SC	20	25	38	0.66	0.01	0.18	0.25	5	20	81	8.3	11.2	27.0	17	11	
63.5	299.2	2.6	3	SC		26			0.02	0.18	0.25	5	20	70	7.9	12.2	31.4	23	14	19.8
66.0	296.7	2.7	2	S		25			0.18	0.19	0.20	11	14	17				29	18	22.4
69.5	294.2	2.8	2	S		25				0.24			13			4.9		43	26	30.9
71.0	291.7	3.1	4	CS	20	27	33	0.83	0.01	0.03	0.10	37	76	85	10.4	23.6	32.0	10	6	
73.5	289.2	3.0	3	SC	18	24	33	0.74	0.01	0.11	0.24	11	49	88	3.9	18.5	33.0	33	19	28.3
76.0	286.7	3.0	1	S		16				2.80			9			2.8		51	29	31.6
78.5	284.2	3.1	2	S		16				2.20		?	?	?				81	46	

B-0-1
Barkley dam

Depth	Elev	EVS	Soil		(--%---			)	(------D50	------	------)	(--P200	--	)	(-----P005	----	)	(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	347.5	1.2	0	_														7	6	
2.5	346.0	0.1	1			22				0.20		15	15	15				7	23	28.6
4.0	344.5	0.2	1	S		24			0.14	0.16	0.20	15	25	28				7	17	23.9
5.5	343.0	0.2	1	S	18	24	30	0.60		0.20			15					6	13	17.3
7.0	341.5	0.3	1	S					0.14	0.16	0.20	15	19	22				9	16	23.7
8.5	340.0	0.3	1	C									56					11	20	29.2
10.0	338.5	0.4	1	C	20	25	32	0.78					93					8	13	
11.5	337.0	0.4	1	C	19	27	35	0.77		0.25			5					10	16	
13.0	335.5	0.4	1			27				0.20			15					7	10	14.5
14.5	334.0	0.5	1	C	18	24	31	0.77		0.20			15					8	11	
16.0	332.5	0.6	1	_		22												9	12	
17.5	331.0	0.6	1	_		26												11	14	
19.0	329.5	0.6	1	C		27				0.25		5	47	56				10	12	20.0
20.5	328.0	0.7	1	C	16	26	26	1.00	0.03	0.12	0.20	15	43	71		19.9		5	6	12.3
22.0	326.5	0.7	1	_		28												4	5	
23.5	325.0	0.8	1	C						0.20			15			81.0		5	6	
25.0	323.5	0.8	1	_		28												6	7	
26.5	322.0	0.9	1	C	18	28	31	0.90					90					4	4	10.5
28.0	320.5	0.9	1	_														5	5	
29.5	319.0	1.0	1	S		26			0.19	0.19	0.19		13					17	17	21.3
31.0	317.5	1.0	1	S						0.17			16					35	35	32.5
34.0	314.5	1.1	1	S						0.25			5					16	15	15.4
35.5	313.0	1.1	1	S					0.16	0.18	0.25	5	15	18				22	21	26.4
37.0	311.5	1.2	1	S						0.25			5					14	13	13.1
38.5	310.0	1.2	1	S	16		24			0.08			48					4	4	9.8

B-0-2

Barkley dam

Depth	Elev	EVS	Soil		(--%---			---)	(------D50		------)	(--P200		--)	(-----P005		----)	(SPT)		(-Mic--)
feet	feet	tsf	%	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
6.5	343.2	0.3	1	S	18	22	24	0.92	0.11	0.13	0.20	15	32	37				5	10	16.2
8.0	341.7	0.3	1	S		27				0.10			41					5	9	15.4
9.5	340.2	0.3	1	S					0.10	0.13	0.20	15	29	36				9	15	22.6
11.0	338.7	0.4	1	C	17	22	31	0.71				81	81	81				8	12	
12.5	337.2	0.5	1	C	17	24	31	0.77					86					7	10	
14.0	335.7	0.5	1	C		26												6	8	
15.5	334.2	0.5	1	C	19	26	38	0.68		0.20			15					7	10	
17.0	332.7	0.6	1	C	17	27	25	1.08					75					9	12	18.9
18.5	331.2	0.6	1	S						0.09			42					7	9	15.2
20.0	329.7	0.7	1	S		28				0.20			15					8	10	13.8
21.5	328.2	0.7	1	C		28				0.20		15	57	65				3	4	9.8
23.0	326.7	0.8	1	C		28							76					3	3	9.7
24.5	325.2	0.8	1	C		31												3	3	
26.0	323.7	0.9	1	C		28												7	8	
27.5	322.2	0.9	1	C	18	29	32	0.91										4	4	
29.0	320.7	1.0	1	C	18	30	30	1.00					79					3	3	9.4
32.0	317.7	1.0	1	S					0.17	0.19	0.25	5	10	11				39	36	32.5
33.5	316.2	1.1	1	S					0.19	0.20	0.25	5	9	10		1.8		30	29	31.5
35.0	314.7	1.1	1	S					0.18	0.20	0.25	5	15	18				22	21	26.1
36.5	313.2	1.2	1	S					0.18	0.20	0.25	5	13	16				25	23	28.0
38.0	311.7	1.2	1	S					0.09	0.13	0.25	5	30	39				18	16	24.0

6-D-3
Barkley dam

Depth feet	Elev feet	EVS tsf	%	Soil ID	PL	Wn	LL	Wn/LL	D50 L	D50 Aver	D50 H	P200 L	P200 Aver	P200 H	P005 L	P005 Aver	P005 H	SPT N	SPT N1	N1c P200
1.0	348.8	0.0	1	M	23	18	30	0.60				84	84	84				20	96	32.5
2.5	347.3	0.1	1	C	20	21	28	0.75					82					9	26	
4.0	345.8	0.2	1	C		24												9	19	
5.5	344.3	0.3	1	C		22												12	22	
7.0	342.8	0.4	1	C		25												9	14	
8.5	341.3	0.5	1	S						0.10			38					19	27	31.9
10.0	339.8	0.6	1	S		18				0.09			44					15	20	28.9
11.5	338.3	0.7	1	C	18	22	31	0.71					80			1.0		8	10	
13.0	336.8	0.7	1	C		25												7	8	
14.5	335.3	0.7	1	C		26												7	8	
16.0	333.8	0.8	1	C	20	26	35	0.74					95					6	7	
17.5	332.3	0.8	1	C		26												9	10	
19.0	330.8	0.9	1	C		27												7	7	
20.5	329.3	0.9	1	S		29							40					11	11	16.6
22.0	327.8	1.0	1	S		27			0.05	0.07	0.20	15	54	60		18.5		5	5	11.3
23.5	326.3	1.0	1	C		27												3	3	
25.0	324.8	1.1	1	C		28										1.0		3	3	
26.5	323.3	1.1	1	C		30												4	4	
28.0	321.8	1.2	1	C	21	31	36	0.86					82					4	4	
29.5	320.3	1.2	1	C		29												2	2	
31.0	318.8	1.2	1	C		30												3	3	
32.5	317.3	1.3	1	S									15					27	24	29.5
34.0	315.8	1.3	1	S						0.25			5					31	27	27.1
35.5	314.3	1.4	1	S					0.14	0.17	0.25	5	19	23				24	21	28.2
37.0	312.8	1.4	1	S						0.25			5					16	13	13.5
38.5	311.3	1.5	1	S					0.19	0.21	0.25	5	14	20		5.0		21	17	22.2

6-2-4

Barkley dam

Depth	Elev	EVS		Soil	(--%---			)	(------D50		------)	(--P200		--)	(-----P005		----)	(SPT)		(-Nlc--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	(--L	--Aver--	-H--)	(L	-Aver-	H)	(--L	--Aver--	H-)	N	N1	P200
1.0	347.5	0.0	1	C		18				0.25			5					15	>99	
2.5	346.0	0.1	1	C	19	19	32	0.59		0.25			5					17	51	
8.5	340.0	0.5	1	C	17	22	30	0.73										11	16	
10.0	338.5	0.5	1	C		22							67					8	11	17.8
11.5	337.0	0.6	1	C		27				0.20			15					2	3	6.4
13.0	335.5	0.6	1	C	17	23	29	0.79										5	6	
14.5	334.0	0.7	1	C		25												5	6	
16.0	332.5	0.7	1	C	17	25	30	0.83		0.01			87			33.0		3	4	
17.5	331.0	0.8	1	C		37						57	57	57				10	11	18.4
20.5	328.0	0.9	1	C		26				0.20		15	42	69				6	6	12.7
23.5	325.0	1.0	1	C		27							68					4	4	10.3
26.5	322.0	1.1	1	C		29							81					4	4	10.1

B-0-5

Barkley dam

Depth	Elev	EVS	Soil		(--%--)				(------D50------)			(--P200--)			(-----P005----)			(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	349.4	0.0	1	C	19	19	49	0.39		0.20		15	62	82				9	51	
4.0	345.4	0.2	1	S		20				0.17			25					13	27	32.4
5.5	343.9	0.3	1	S					0.17	0.18	0.20	13	14	15				8	15	19.4
7.0	342.4	0.4	1	C	17	22	28	0.79					64					9	15	
8.5	340.9	0.4	1	S		25				0.10			39					11	16	26.4
10.0	339.4	0.4	1	S		23												8	12	12.0
11.5	337.9	0.5	1	C		25												8	12	
13.0	336.4	0.5	1	C	18	26	34	0.76										8	11	
14.5	334.9	0.6	1	C		29										1.0		5	7	
16.0	333.4	0.6	1	C	17	25	33	0.76										6	8	
28.0	321.4	1.0	1	C	19	29	30	0.97					79					4	4	10.2
29.5	319.9	1.0	1	C		27												4	4	
31.0	318.4	1.1	1	C		27												5	5	
32.5	316.9	1.1	1	S					0.24	0.24	0.25	5	6	6				26	25	26.1
34.0	315.4	1.2	1	S					0.23	0.23	0.25	5	7	7				25	23	24.7
35.5	313.9	1.2	1	S						0.25			5					20	18	18.4
37.0	312.4	1.2	1	S						0.25			5					18	16	16.2
38.5	310.9	1.3	1	S						0.25			5					16	14	14.2

S-D-6

Barkley dam

Depth	Elev	EVS	Soil*		(--%---)				(------D50------)			(--P200--)			(-----P005----)			(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	* P200
1.0	347.6	0.0	1	C	18	22	28	0.79		0.20			15					8	59	
2.5	346.1	0.1	1	C		22				0.25			5					15	49	
4.0	344.6	0.2	1	M	18	23	21	1.10										8	19	18.6
5.5	343.1	0.2	1	M						0.20			15					4	8	12.3
7.0	341.6	0.3	1	C	18	24	51	0.47										11	21	
8.5	340.1	0.3	1	C		23												8	14	
10.0	338.6	0.4	1	S						0.09			43					11	18	26.1
11.5	337.1	0.4	1	S						0.08			49					12	19	27.5
13.0	335.6	0.5	1	C	17	22	26	0.85										10	15	
14.5	334.1	0.5	1	C		23												12	17	
16.0	332.6	0.6	1	C	19	25	30	0.83										6	8	
17.5	331.1	0.6	1	C		24												5	6	
19.0	329.6	0.7	1	C		22							57					7	9	15.1
20.5	328.1	0.7	1	C		24												9	11	
22.0	326.6	0.7	1	S	17	28	27	1.04		0.08			49					6	7	13.3
23.5	325.1	1.2	0	_														3	3	
25.0	323.6	0.8	1	C	17	30	24	1.25		0.03			75			21.0		5	5	
26.5	322.1	0.9	1	C	18	27	24	1.13					89					4	4	10.5
28.0	320.6	0.9	1	C	17	27	27	1.00		0.02			67			24.0		4	4	
29.5	319.1	1.0	1	C		30						87	87	87				6	6	12.3
31.0	317.6	1.0	1	C	19	32	37	0.86		0.03			72			23.0		5	5	
32.5	316.1	1.1	1	C		31				0.20			15					3	3	6.7
34.0	314.6	1.1	1	C		26												6	6	
35.5	313.1	1.1	1	C	20	28	35	0.80		0.20			15					5	5	
37.0	311.6	1.2	1	C		29												7	6	
38.5	310.1	1.2	1	C	18	31	35	0.89		0.02			87			26.5		5	4	

B-D-7

Barkley dam

Depth	Elev	EVS	Soil		(--%---)				(------D50------)			(--P200--)			(-----P005----)			(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	347.0	0.0	1	C		17				0.25			5					25	>99	
2.5	345.5	0.1	1	C	22	28	49	0.57										13	37	
4.0	344.0	0.2	1	C		28												10	21	
5.5	342.5	0.3	1	C	19	25	51	0.49										13	23	
7.0	341.0	0.4	1	C		26												10	16	
8.5	339.5	0.5	1	C		23												13	18	
10.0	338.0	0.5	1	C		23												9	12	
11.5	336.5	0.6	1	C	17	22	32	0.69										10	13	
13.0	335.0	0.6	1	C		24												9	11	
14.5	333.5	0.7	1	C		22												11	13	
16.0	332.0	0.7	1	C		24												6	7	
17.5	330.5	0.8	1	S		27												9	10	10.2
19.0	329.0	0.8	1	C		24												6	7	
20.5	327.5	0.9	1	C		27												3	3	
22.0	326.0	0.9	1	S		23				0.08			49			14.5		6	6	12.5
23.5	324.5	1.0	1	S						0.25			5					13	13	13.5
25.0	323.0	1.0	1	S					0.16	0.18	0.25	5	29	35		12.0		16	16	23.6
26.5	321.5	1.0	1	S						0.25			5					17	17	16.9
29.0	320.0	1.1	1	C	18		26											4	4	
29.5	318.5	1.1	1	S														21	20	19.7
31.0	317.0	1.2	1	S						0.25			5					25	23	23.3
32.5	315.5	1.2	1	S					0.21	0.22	0.25	5	9	10				25	23	25.4
34.0	314.0	1.3	1	S						0.25			5					26	23	23.3
35.5	312.5	1.3	1	C		22				0.25			5					14	12	12.3
37.0	311.0	1.4	1	C		26			0.07	0.13	0.25	5	38	54		16.0		14	12	19.4
38.5	309.5	1.4	1	C		29												4	3	

[illegible]

[illegible] dam

Depth	Elev	CVS		Soil	(--%---			)	(------	D50	------)	(--	P200	--)	(-----	P005	----)	(SPT)		(-Nlc--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	347.6	0.0	1	C	22	19	34	0.56					88					13	74	
2.5	346.1	0.1	1	C	22		49						94					19	54	
4.0	344.6	0.2	1	C		26												17	36	
5.5	343.1	0.3	1	C		24												14	25	
7.0	341.6	0.4	1	C														20	31	
8.5	340.1	0.5	1	C	18	21	23	0.91		0.07			51					15	21	30.5
10.0	338.6	0.6	1	C		21			0.06	0.12	0.25	5	39	56		19.5		10	13	21.0
11.5	337.1	0.6	1	C	18		29						92					7	9	15.4
13.0	335.6	0.7	1	C		26												3	4	
14.5	334.1	0.7	1	C		25												6	7	
16.0	332.6	0.8	1	C	17	25	33	0.76		0.20		15	52	89				7	8	
17.5	331.1	0.8	1	C		25												6	7	
19.0	329.6	0.8	1	S		27			0.14	0.16	0.25	5	11	12				13	14	17.1
20.5	328.1	0.9	1	C						0.05			63					9	10	16.1
22.0	326.6	0.9	1	C	17	27	25	1.08		0.02			74			21.5		3	3	
23.5	325.1	1.0	1	C		28												4	4	
25.0	323.6	1.0	1	C	19	27	30	0.90					96					5	5	
26.5	322.1	1.1	1	C														7	7	

B-0-9

Barkley dam

Depth	Elev	EVS		Soil	(--%---)				(------D50	------)		(--P200	--)		(-----P005	----)		(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
2.5	371.5	0.1	1	C	18	20	46	0.43										18	51	
4.0	370.0	0.2	1	C		18												16	34	
5.5	368.5	0.3	1	C		22												8	14	
7.0	367.0	0.4	1	C	18	19	28	0.68										20	30	
10.0	364.0	0.6	1	C		19				0.25			5					22	29	29.7
11.5	362.5	0.7	1	C		19												22	27	
13.0	361.0	0.8	1	C		22												19	22	
14.5	359.5	0.9	1	C	16	17	30	0.57					71					25	27	
16.0	358.0	1.0	1	C		21				0.25			5					21	21	21.5
17.5	356.5	1.0	1	C		20				0.25			5					13	13	13.1
19.0	355.0	1.1	1	C		18												14	13	
20.5	353.5	1.2	1	C		18												20	18	
22.0	352.0	1.3	1	C		19				0.25			5					27	23	23.7
23.5	350.5	1.4	1	C		19												23	19	
25.0	349.0	1.5	1							7.50			11					67	54	32.5
26.5	347.5	1.6	1						0.25	6.53	7.50	5	11	12				15	12	14.6
28.0	346.0	1.7	1	C	18	18	41	0.44										22	17	
29.5	344.5	1.7	1	C		18				0.25			5					30	23	23.1
31.0	343.0	1.8	1	C		24												18	13	
32.5	341.5	1.8	1	C		22												19	14	
34.0	340.0	1.9	1	C		19												17	12	
35.5	338.5	1.9	1	S						0.13			28					28	20	29.5
37.0	337.0	2.0	1	S		22				0.25			5					15	11	10.9
38.5	335.5	2.0	1	C	18	22	33	0.67										10	7	
40.0	334.0	2.1	1	C		22												12	8	
41.5	332.5	2.1	1	C		22												11	8	
43.0	331.0	2.1	1	C		25				0.20			15					7	5	8.6
44.5	329.5	2.2	1	S		24				0.09			32					22	15	22.5
46.0	328.0	2.2	1	S		25				0.08			38					14	9	16.0
47.5	326.5	2.3	1	M		25							60					11	7	13.6
49.0	325.0	2.3	1	C	17	27	27	1.00										7	5	
50.5	323.5	2.4	1	C		27												9	6	
52.0	322.0	2.4	1	C		25												12	8	
53.5	320.5	2.5	1	C		28												7	4	
55.0	319.0	2.5	1	C	19	24	31	0.77		0.25			5					10	6	
56.5	317.5	2.6	1	C		25												8	5	
58.0	316.0	2.6	1	C		27												6	4	
59.5	314.5	2.7	1	C		28												7	4	
61.0	313.0	2.7	1	C		24				0.07			52					13	11	17.9
64.0	310.0	2.7	0							0.20			15					5	3	

ES-3

Barkley dam

Depth	Elev	EVS	Soil*		(--Z---)				(------D50------			)(--P200--		)(	-----PDGS----		)(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
7.0	333.0	0.4	1	C	22	21	44	0.49		0.01		92	92	92		41.0		15	25	
13.0	327.0	0.7	1	C	20	20	41	0.48		0.01			85			39.1		23	27	
25.0	315.0	1.4	1	C	16	16	30	0.53		0.03			45		25.7	25.7	25.7	5	4	
69.3	270.7	2.7	1	S						0.30			7			3.6		27	16	17.7
75.0	265.0	2.9	1	S	15	15	30	0.49		0.28			32			14.8		10	6	11.9

Waterways Experiment Station Cataloging-in-Publication Data

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